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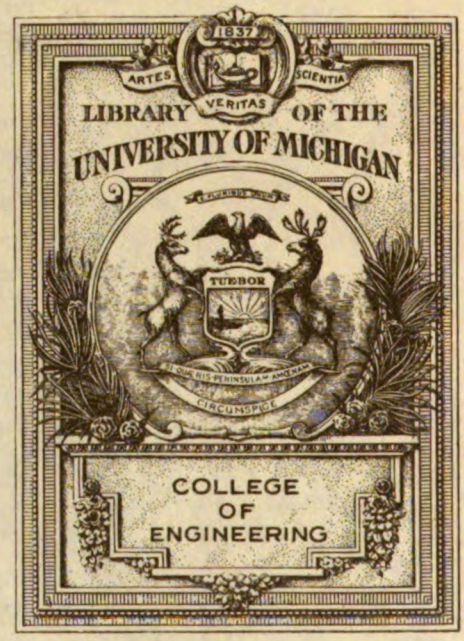

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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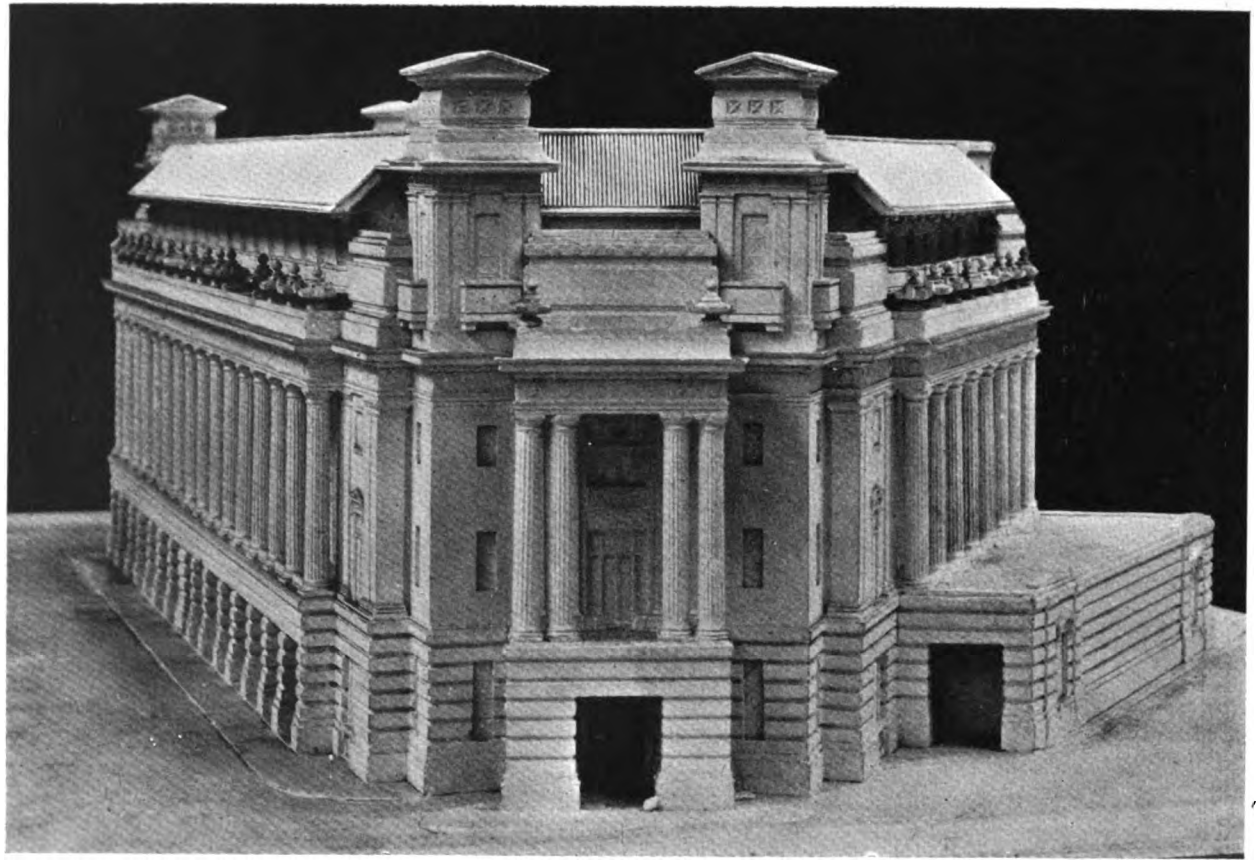

[*Major P. H. Keys, D.S.O., F.R.I.B.A., Architect.*

NEW GENERAL POST OFFICE BUILDING, SINGAPORE. (*See p.* 23.)

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume XX. No. 1. LONDON, JANUARY, 1925

EDITORIAL NOTES.

1924 AND AFTER.

At this time of the year it is our pleasant privilege to wish all our readers a very happy and prosperous New Year and to take an optimistic view of the future, relegating to the back of our mind, where they now belong with other things of the past, the trials and difficulties which beset the building industry during the past year. We think of the enormous amount of work held up on account of cost and skilled labour shortage, and look forward to the time when all this building gets beyond the drawing-board stage and the boom which must result is with us. We forget last year's national building strike, and hope the present high level of building wages as compared with those ruling in other trades will induce the operatives to give of their best so that this extra cost at any rate may be credited on the other side of the balance sheet. Our irksome and anomalous building by-laws are out of mind for the time being, and we are hopeful that when Mr. Topham Forrest at the end of the month produces his report on the American methods he recently inspected, the London County Council and other responsible bodies will revise their regulations so that the best advantage may be taken of materials and methods now available. We had our usual annual Housing Act last year, and there are now two distinct Acts on the statute book under which subsidies are granted for small houses; if only private enterprise were given the same encouragement as local authorities, and rent restriction modified, we could almost predict the return to business of the speculative builder on whom we previously relied for this class of property. However, we console ourselves with the fact that over 120,000 houses were built last year. We dismiss the frequent uninformed statements, made by those unacquainted with a modern concrete house, that concrete houses are unsatisfactory, and remember that the Chief Architect of the London County Council tells us he intends to make much greater use of concrete for small house construction in the future. During the past year concrete had at Wembley the finest advertisement that could be conceived, and we hope the attention these buildings received from the architectural world and the public will lead to a greater interest and use of concrete for architectural and general use in the future. At the end of the year the new rapid-hardening Portland cement was put on the market as a commercial proposition, and in addition to its obvious advantages where speed is any consideration at all, it should lead to a much wider field of usefulness for concrete.

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Those engaged in building and constructional work have to face many difficulties peculiar to themselves, but a spirit of hopefulness, a spirit which is, we think, amply warranted by the future outlook, will go a long way to overcome them. If the stable Government we are now promised leads to stability in industry and national life, and a gradual return to whatever the long delayed post-war normal conditions may prove to be, we have no fear for the future or of the development of the concrete industry.

DEVELOPMENTS IN CEMENT MANUFACTURE.

THE article in this issue dealing with practical tests of a quick hardening cement calls attention to one of the most remarkable developments that has occurred in the history of Portland cement. The advent of aluminous cement and the proof it has given of the value of the rapid hardening of concrete has apparently spurred the manufacturers of Portland cement to fresh efforts, and there is now on the market a true Portland cement which gives 24-hour tests higher than those demanded by the British Standard Specification at 28 days, and which will produce a concrete of the usual 1 : 2 : 4 mixture possessing at 24 hours more than the strength demanded by the London County Council at 28 days.

Laboratory tests of cement are frequently regarded with suspicion by those who are inclined to pride themselves as being practical men, and we are therefore glad to observe that the tests reported in the article referred to have been on a practical scale. The correspondence between the laboratory tests and the practical tests is satisfactory and should be useful in securing confidence in laboratory tests.

The production of cement of such improved rapid hardening qualities should give an impetus to the use of concrete for roads and piles, because the previous objections of delay in hardening can no longer be valid. It is satisfactory to know that quick hardening Portland cement need not cause any misgivings as to its ultimate stability, because it is produced by no other means than refinements in the process of manufacture, and that this is realised by engineers is evident from the fact that the new rapid-hardening cement is already being used on important works, such as the new Albert Bridge at Windsor, illustrated and described in our last issue, and the reconstruction of Piccadilly now proceeding.

QUANTITY OF WATER IN CONCRETE.

As is now well known, the strength of concrete depends as much on the quantity of water as it does on the quantity of cement, and therefore it is always more economical to get the maximum strength by using the correct quantity of water than by using an excess of cement. Too little water in concrete has the effect that the particles of sand and ballast are not sufficiently well lubricated to slip into place in such a way as to get a concrete of maximum density. The particles remain touching on points, with considerable air voids, and form a relatively porous concrete, and if the lack of water is carried to excess there may even be insufficient water to develop the full chemical action of which the cement is

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capable. In exactly the same way an excess of water is detrimental because the concrete which with the minimum of water might occupy 1 cu. ft., for example, will with a certain excess of water occupy, say, 1·2 cu. ft. In course of time the excess water evaporates and its place is taken by air, so that once more the concrete is left relatively porous with the particles touching on points instead of over their whole area. It is quite clear that the densest concrete will be the strongest, and this has been proved by numerous experiments.

The matter is not of theoretical interest only, and cannot be ignored by the practical man. This is made sufficiently clear when it is mentioned that if in a certain concrete 10 gallons of water per bag of cement give the best results, the loss of strength may amount to 50 per cent. if 15 gallons are used. In reinforced concrete it happens that the percentage of water which gives the greatest strength gives too dry a mixture for practical purposes. It is clear that it is one thing to tamp a dry concrete mixture into a cast-iron mould for making test cubes and quite another thing to try to use the same dry concrete in a reinforced concrete beam, where the concrete not only has to be inserted and rammed between the reinforcements, but also has to be contained inside a wooden mould which is not capable of resisting great lateral pressure. In a reinforced concrete beam it is often more important to make certain that the rods will be everywhere surrounded with concrete, without large air pockets and other defects, than to ensure the concrete being of the maximum possible strength. In other words, a slightly weaker concrete is better than no concrete at all.

It follows, therefore, that in reinforced concrete work a compromise has to be made between the concrete of maximum strength, which is a very dry mixture, and a concrete which can be used in practice without the risk of considerable air voids around the reinforcing rods. It is also clear that it is as important to control the quantity of water as it is to control the quantity of cement. We have already given tables prepared by American authorities on this subject, and in this issue give a table prepared by Dr. Oscar Faber, O.B.E., D.Sc., showing the quantity of water which he uses for different concretes. We are sure a table such as this will be of value to all concerned with the concrete industry, providing as it does a ready means of ascertaining the amount of water required to obtain the best mixture for different purposes.

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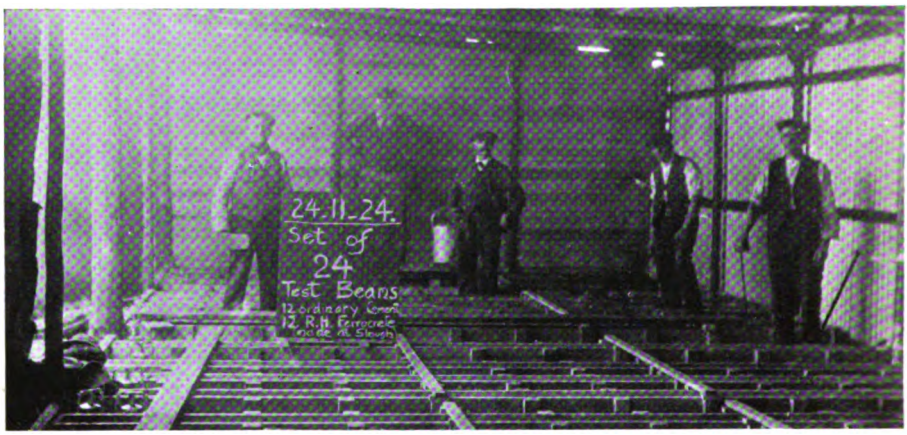

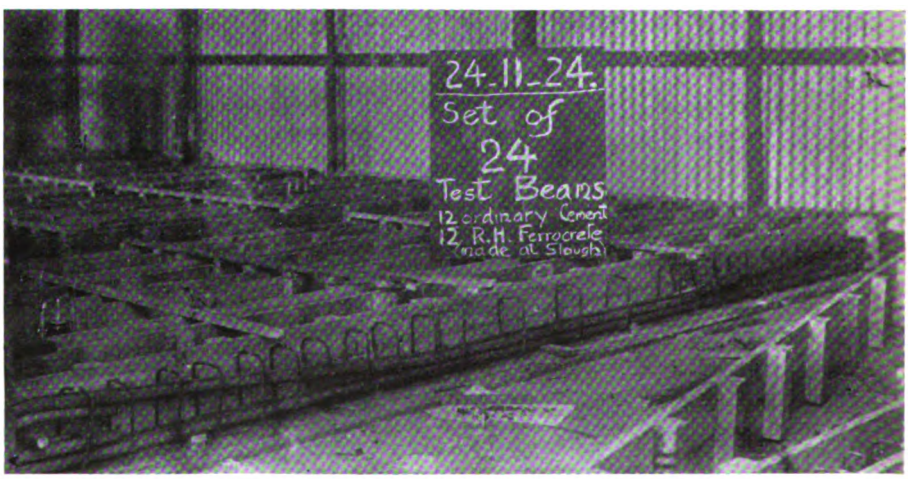

RAPID-HARDENING PORTLAND CEMENT.

[Showing practical conditions under which the test beams were made.] (*See p.* 5.)

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RAPID HARDENING PORTLAND CEMENT.

Results of Practical Tests.

THE relation between the laboratory tests of a cement and the strength of concrete members of practical dimensions containing that cement has frequently been the subject of divided opinion. There was perhaps good reason for this in the days before the British Standard Specification for Portland Cement when test briquettes could be rammed in such a way as to produce a 50 per cent. higher test result than when the cement was filled into the moulds in a plastic state.

But even now there are on the one hand the cement manufacturers who present tensile tests of their products showing that modern cement has double the strength of its predecessor of 10 or 20 years ago, and who urge engineers to avail themselves of this development by increasing their working stresses, and on the other side there are the engineers who by continuing to restrict themselves to a working stress of 600 lbs. per sq. in. in the concrete imply that they have not sufficient confidence in laboratory tests to adopt their logical application in the design of structures.

This position has repeated itself in connection with the rapid hardening Portland cements, which are the latest development of the cement industry. The manufacturers assert, with evidence in the shape of laboratory tests, that rapid hardening cement possesses at 2 days the strength usually not expected before 28 days, but users are slow to put this assertion to the test by driving piles at an age of 2 days or removing supports from suspended floors at the same period.

The manufacturers of "Ferrocrete"—one of the brands of rapid hardening cement—have taken steps to bridge the gap between laboratory tests and practical usage by arranging for concrete beams of dimensions common in structural work to be made by a contractor under ordinary working conditions and then tested to destruction under independent auspices. The details of the testing are given herewith, and it will be observed that the laboratory tests give a true indication of the concrete-making qualities of the cement and of its rate of hardening. This conclusion, important as it is, seems, however, of small moment compared with the remarkable strength shown by the concrete. That a 6 in. by 10 in. concrete beam of 15 ft. span would carry more than 6 tons at an age of 2 days would have been incredible a few years ago. A load of 10 tons at 7 days is no less remarkable. In the absence of any standard beam test comparative tests were made with an ordinary commercial Portland cement, and it will be seen that within the period covered by the tests the concrete made with the "Ferrocrete" (Rapid Hardening) Portland cement is from two to three times as strong as that made with ordinary cement, although the latter, as shown by the test of 432 lbs. per sq. in. at 7 days, when mixed with 3 parts of sand has double the strength demanded by the British Standard Specification.

Starting with the knowledge that the compressive strength of the concrete made with the "Ferrocrete" (Rapid Hardening) Portland cement ranged from 2,500 lbs. per sq. in. at 2 days to 7,000 lbs. per sq. in. or more at 28 days, it was obvious that the testing of concrete beams of ordinary design would be useless to demonstrate the true strength of the concrete apart from the reinforcement. In a reinforced concrete beam designed for the usual working stress of 600 lbs.

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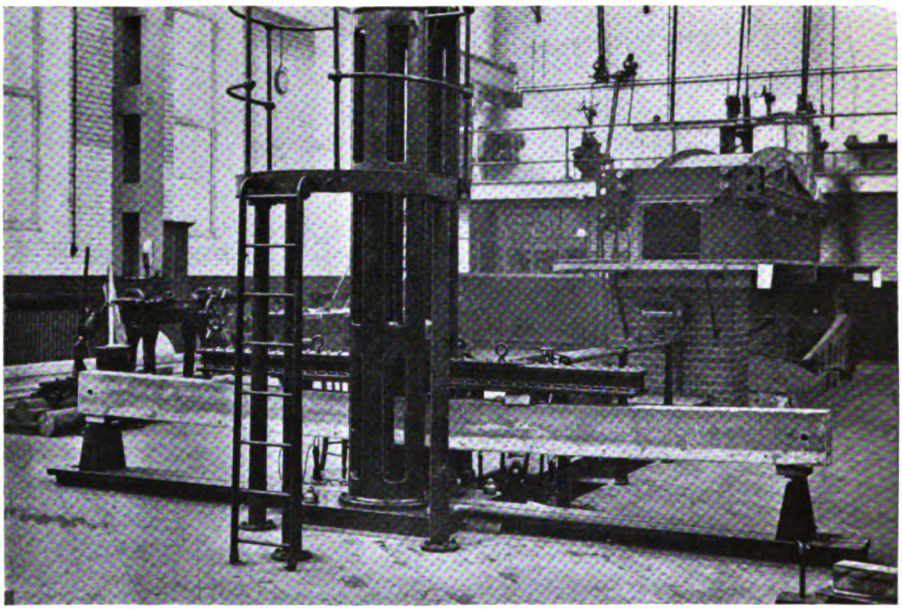

TEST IN PROGRESS.

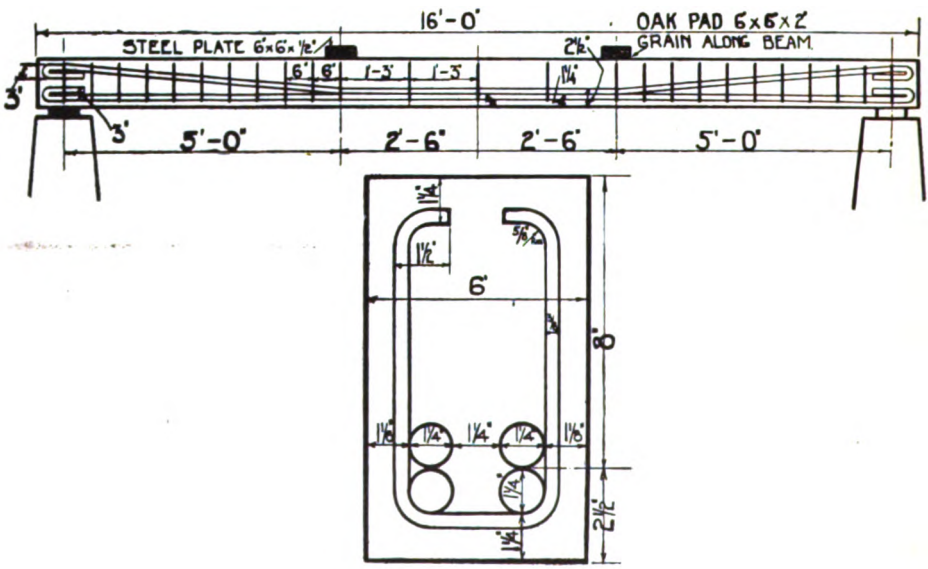

FIG. 1.
DESIGN OF TEST BEAMS.

TESTS ON RAPID HARDENING PORTLAND CEMENT.

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per sq. in. in the concrete the beam made of the concrete just described would fail under a breaking load on the tension side even at an age of 2 days, because the strength of the concrete in the compression side of the beam would be so much greater than that of the steel in the tension side. Hence for the purpose of showing the compressive strength that the special concrete possessed at ages of 2 days and upwards it became necessary to make beams of special design in which there would be sufficient reinforcement to prevent failure, under breaking loads, due to tension. The design of the beams was undertaken by Dr. Oscar Faber, O.B.E., D.Sc., who also drew up the specification for the concrete and the casting and testing of the beams.

The dimensions and design of the beams are shown in *Fig.* 1, and Dr. Faber's specification, which was strictly adhered to, is as follows :—

SPECIFICATION OF TEST BEAMS TO SHOW THE COMPARATIVE STRESS IN ACTUAL PRACTICE OF ORDINARY CEMENT AND "FERROCRETE" (RAPID HARDENING) PORTLAND CEMENT.

Twenty-four beams are to be made in strict accordance with drawing No. F. 1386 (*Fig.* 1). Twelve of these beams are to be made with ordinary cement and are to be marked "A" and the remainder are to be made with "Ferrocrete" (Rapid Hardening) Portland cement and marked "B." The beams are to be numbered numerically "A 1" to "A 12" and "B 1" to "B 12." All beams are to be made on the same day under identical conditions and stored under identical conditions.

Concrete.—The concrete is to consist of :—

Ham River ballast	8 cu. ft.
passing ½ in. retained on $\frac{1}{16}$ in. screen	
Ham River sand	4 cu. ft.
Cement	One bag of 204 lbs.
Water	10 gallons.

Water.—The quantity of water above given is based on the assumption that the sand and ballast are practically dry. If, however, they are wet or moist the quantity of water must be reduced so as to give the same consistency of concrete as is obtained with 10 gallons using dry materials. As it is important that the test shall be strictly comparative exactly the same quantity of water must be used for all beams, and the sand and ballast must be of uniform dryness throughout. If it is found that part of the bulk is appreciably drier than the rest it must be mixed before the concreting of the beams is begun so as to ensure that we are dealing with uniform materials.

Mixing.—The mixing should preferably be done with a small mixing machine of the batch type, but if done by hand should be turned over three times dry and three times wet.

Placing.—Before placing the concrete, the inside surface of the moulds should be coated with soft soap and whiting. Half-inch of cement mortar, consisting of

Ham River sand	4 cu. ft.
Portland cement	One bag of 204 lbs.
Water	7½ gallons

should be placed at the bottom of the mould first and ordinary concrete immediately placed thereon. The concreting of one beam must proceed uninterruptedly until it is finished in one operation.

Steel.—The steel rods are to be mild steel to British Standard Specification. To ensure duly comparative results all the rods must be from the same rolling. They are to be bent strictly to the shape shown on the drawings. The hooks at the end may be heated during bending, but the bends towards the middle must be done cold. They are to be scraped clean of all loose scale with a wire brush.

Cover of Concrete.—It is of the utmost importance that the rods are exactly the right distance from the bottom of the beam, especially for the middle half of the beam. Special precautions must be taken to ensure this, and it is suggested that this could be done by passing a ⅜-in. rod through two holes in the sides of the centering box on which to rest the rods at the right level, such ⅜-in. rods being provided at 5 ft. centres along the beam. These rods may be removed as soon as the whole of the concrete is in place and before the concrete sets.

Testing of Steel.—The steel used for the

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main rods and for the stirrups should for purposes of record be tested.

Test Cubes of Concrete.—Twenty-four cubes of the actual concrete used in the beams should be made in 6-in. steel or cast-iron moulds for testing purposes and numbered and lettered the same as the beams which they represent, "A 1" to "A 12" and "B 1" to "B 12."

Handling.—The test beams during handling may only be supported at their ends. It is suggested that a 1-in. hole be left about half-way up the beam and 4 in. from each end to facilitate slinging.

Centering.—The centering should be made from 1¼-in. wrought timber, and cramps should be provided at the two-third points to prevent the pressure of the concrete bulging the form outwards, and care should be taken that the exact dimensions of the beams are maintained. The sides shall be removed the day after concreting, but the beams should be left on their bottoms until required for testing.

Testing.—Two beams of each kind, "A" and "B," are to be tested at the following ages :—

1 and 2	2 days
3 and 4	4 days
5 and 6	7 days
7 and 8	14 days
9 and 10	28 days
11 and 12	3 months.

The corresponding cubes are to be tested at the same periods. During testing, the beams are to rest on two pads of oak 6 in. by 6 in. by 2 in. with the grain along the direction of the beam, placed at 15-ft. centres, and the load is to be applied on two pads of oak 6 in. by 6 in. by 2 in., surmounted by steel plates 6 in. by 6 in. by ½ in. placed 5 ft. from the end bearing and 5 ft. apart.

Great care must be taken to see that the load is equally distributed between the two loading points and is concentric on the beam. It must be applied gradually and without jerks or swinging, and the loads to be recorded are :—

(*a*) The appearance of the first crack.
(*b*) The maximum load which the beam carries before failure.

It is possible that these two results will coincide.

The beams were cast by Messrs. A. Jackaman & Son, Ltd., at their yard at Slough on November 24, 1924, the work being done by ordinary concreters. Messrs. Jackaman's report is as follows :—

Ballast and Sand.—Stored on iron sheets on ground and kept covered for two days before using.

Timber Moulds.—1¼ in. thick with 3 in. by 2 in. battens ; nails clenched on back. Base of beam resting on 1¼ in. thick wrot. fir timber floor laid on 9 in. by 3 in. fir beds truly levelled up. Moulds well secured with battens before, during, and after concreting, until stripping done.

Water. — Slough Urban District Council's main water used, at a temperature of about 60 degrees.

Mixing.—By hand, on clean boarded platforms adjacent to moulds, transported to moulds in buckets. All platforms well washed and scrubbed when gaugings changed from ordinary cement to "Ferrocrete" (Rapid Hardening) Portland cement or *vice versa*.

Situation of Beams during Making.—Inside a shed, three sides and the top of which were enclosed with corrugated iron, the fourth being in movable sections allowing ingress to the part upon which work was being done, all being enclosed after the work was finished save an adjustable panel for temperature regulations. Heat throughout the period of concreting, and subsequently, was supplied by two or more coke braziers, an oil lamp, and 12 hurricane oil lamps. At the age of 24 hours all concrete was stripped and watered twice a day. The temperature is and will be maintained at about 60° F.

The beams were removed from the shed at Slough and transported by lorry when required for testing to the City & Guilds Engineering College at South Kensington, where the tests were conducted under the supervision of Professor S. M. Dixon in the presence of a number of Borough Engineers and others.

In Table 1 will be found the usual laboratory tests of "Ferrocrete" (Rapid Hardening) Portland Cement and ordinary commercial Portland cements with which it was compared in the beam tests.

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TABLE 1.—LABORATORY TESTS.

	Fineness. Residue on sieve of 32400 meshes per sq. inch.	Setting Time. Initial. hr.	Initial. mins.	Final. hrs.	Final. mins.	Soundness. Le Chatelier Test. 24 hours.	Tensile Tests in lbs. per sq. inch. 3 parts Sand, 1 part Cement. 1 day.	2 days.	3 days.	5 days.	7 days.
"Ferrocrete" (Rapid-Hardening) Portland Cement . .	0·3%	1	10	2	20	1 mm.	312	516	583	611	688
Ordinary Cement .	6·05%	1	50	3	30	1 mm.	117	218	354	405	432

COMPRESSION TESTS IN LBS. PER SQUARE INCH—6-IN. CUBES.

4 parts Ham River Ballast, 2 parts Ham River sand, 1 part cement.

	1 day.	2 days.	3 days.	5 days.	7 days.	28 days
"Ferrocrete" (Rapid Hardening) Portland Cement . . .	1,750	2,910	3,853	4,840	5,360	6,810
Ordinary Cement . . .	—	—	—	—	2,860	4,440

Extracted from reports by Messrs. Riley Harbord & Law dated October 21 and November 19, 1924.

Table 2 is an extract from Professor Dixon's report showing the results obtained on testing the beams up to 14 days. Results of later tests will be given as they become available.

TABLE 2.—TESTS OF REINFORCED CONCRETE BEAMS.

Cement.	Age of Beam.		Dimensions. Width.	Depth.	Weight in lbs.	1st Compression Crack—lbs.	Maximum Load in lbs.
	hrs.	min.	in.	in.			
"Ferrocrete" (Rapid-Hardening) Portland	52	15	6·05	10·5	1,283	13,970	13,970
Ditto	53	17	6·2	10·51	1,291	14,380	14,990
Ordinary	53	50	6·19	10·58	1,292	3,700	3,700
Ditto	54		6·26	10·50	1,281	4,130	4,130
	days.	hrs.					
"Ferrocrete" (Rapid Hardening) Portland	4	3·3	6·19	10·58	1,291	15,580	19,680*
Ditto	4	4·15	6·11	10·55	1,270	18,550	18,600
Ordinary	4	4	6·12	10·50	1,269	6,250	6,350
Ditto	4	4·25	6·23	10·55	1,281	6,730	6,860
"Ferrocrete" (Rapid Hardening) Portland	7	3	6·16	10·55	1,285	18,170	20,900
Ditto	7	3·45	6·36	10·64	1,282	22,195	22,545
Ordinary	6	23·25	6·16	10·59	1,270	9,260	9,260
Ditto	7		6·24	10·64	1,285	7,415	7,415
"Ferrocrete" (Rapid-Hardening) Portland	14		6·10	10·58	1,275	24,350	24,500
Ditto	14		6·17	10·67	1,281	23,990	24,240
Ordinary	14		6·27	10·70	1,283	12,140	12,140
Ditto	14		6·18	10·57	1,275	12,680	12,710

* In our last issue this figure was wrongly given as 1,900.

These results tell their own story, and it need only be added that the "Ferrocrete" (Rapid Hardening) Portland cement is a true Portland cement having all the safeguards attached to a building material which has just attained its centenary.

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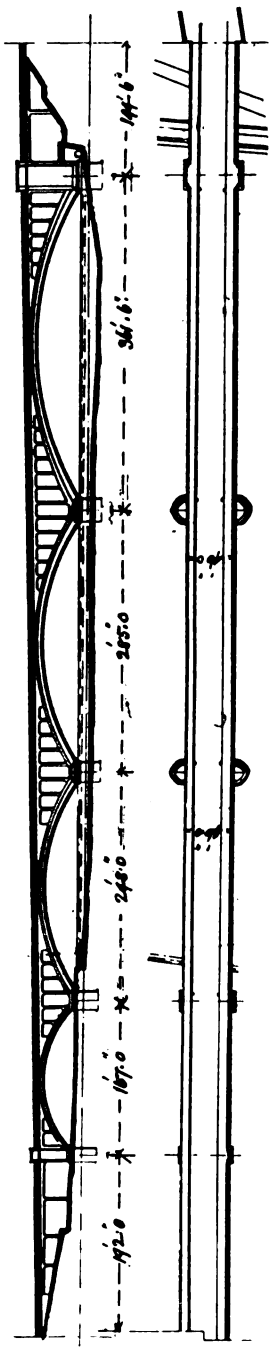
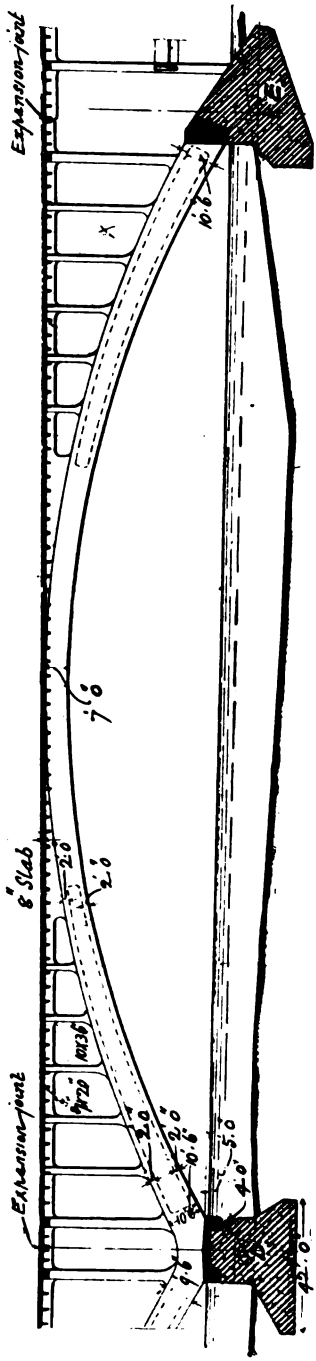


East Elevation and Plan.



Elevation of Main Span.

NEW REINFORCED CONCRETE BRIDGE OVER THE TWEED. (*See p.* 11.)

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NEW REINFORCED CONCRETE BRIDGE OVER THE TWEED.

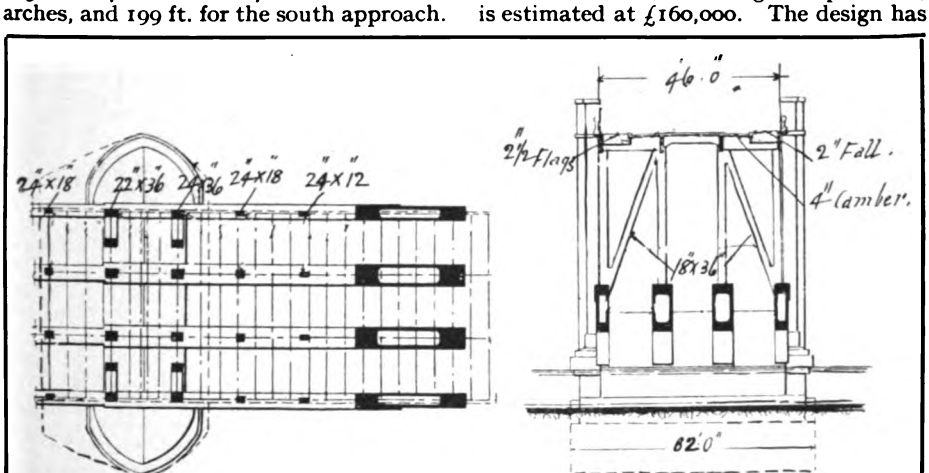
Longest Reinforced Concrete Arch in Great Britain.

We illustrate herewith the design of a new reinforced concrete bridge to be built across the river Tweed at Berwick. The bridge will be 46 ft. wide between parapets, including a roadway of 30 ft. and two footpaths of 8 ft. each. The rock and gravel formation below the bed of the river provide good foundations for the piers, cofferdams for which are now being sunk. At highwater the headroom below the main river arches will be 46 ft. As is shown in the drawing, there will be five piers, four arches and two approach viaducts, the lengths of which will be, from north to south, 144 ft. 6 in. for the north approach, 361 ft. 6 in., 285 ft., 248 ft. and 167 ft. for the four arches, and 199 ft. for the south approach.

bridge and hollow from the centre third to the springing points. The deck will be supported on vertical reinforced concrete supports from the arches. The deck will consist of an 8 in. reinforced concrete slab with transverse beams 7 in. wide by 20 in. deep. Longitudinal beams connecting the vertical supports will be 10 in. wide by 30 in. deep. The position of the expansion joints is shown in the drawing.

The ribs of the main arch will be 10 ft. 6 in. deep at the springing and 7 ft. deep at the crown, by a width of 3 ft. 6 in. for the outer ribs and 5 ft. for the inner ribs. The spans have been designed for a moving load of 1,350 tons.

The total cost, including land purchase, is estimated at £160,000. The design has



Sectional Plan. Section of Main Span.

New Reinforced Concrete Bridge over the Tweed.

The longest span of this bridge, 361 ft. 6 in., will be the longest reinforced concrete bridge arch so far built in Great Britain, and a detail drawing of this span is also given. There will be four arch ribs, solid in the centre third of the

been prepared by Messrs. L. G. Mouchel & Partners, while Mr. J. A. Beard, County Surveyor of Northumberland, is the engineer representing the local authorities concerned. The contractors are Messrs. Holloway Bros. (London), Ltd.

The Stadium at Colombes.

The stadium built at Colombes, near Paris, for last year's Olympic Games, illustrated in our last issue, was designed and erected on the Coignet system by The Constructions Edmond Coignet, of Paris. The cost of the work, which is entirely constructed of reinforced concrete with the exception of the roof of the stand, was about seven million francs.

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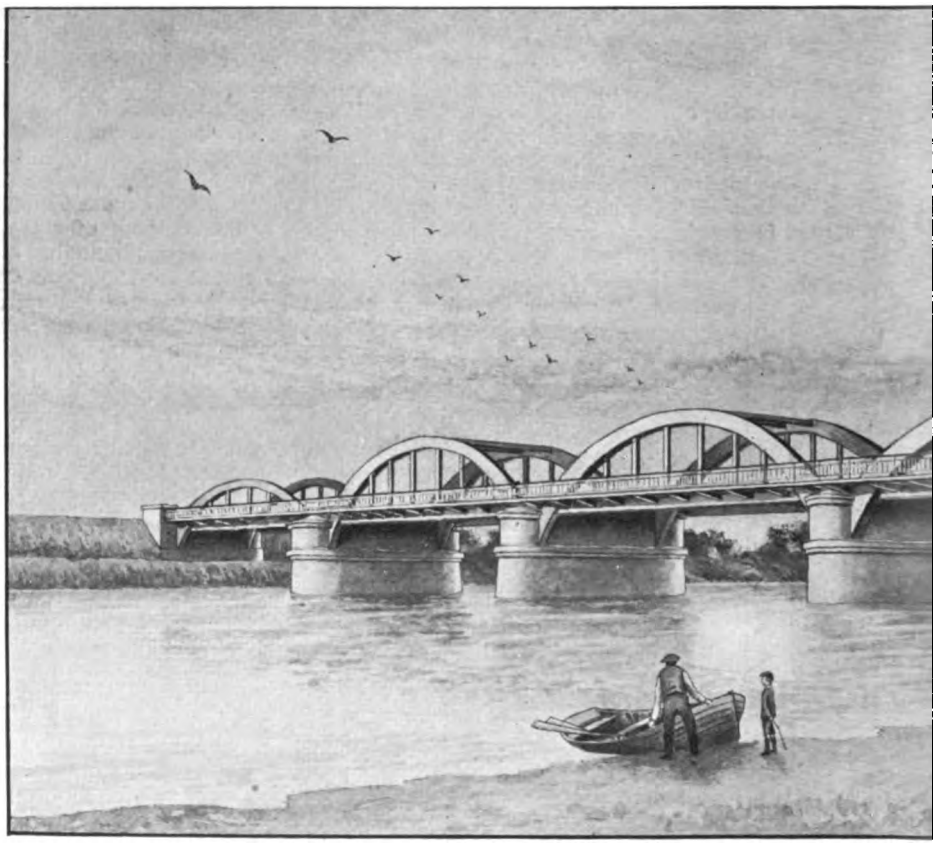

REINFORCED CONCRETE BR

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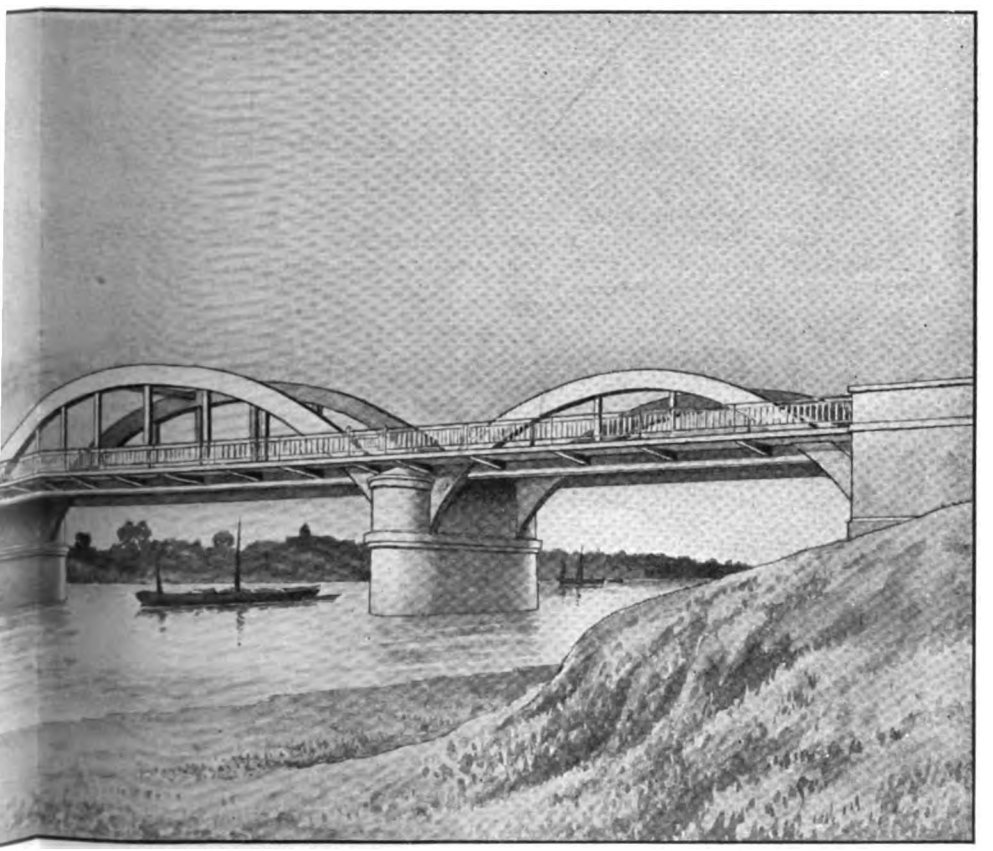

BRIDGE AT KING'S LYNN. (*See p.* 14.)

[*Mr. S. H. Warren, A.M.Inst.C.E., Norfolk County Surveyor, Engineer.*

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NEW REINFORCED CONCRETE BRIDGE AT KING'S LYNN.

THE attempted reclamation of further portions of the Wash draws attention to a part of England which, although little known generally, is of as much interest as the drainage system of Holland, which yearly attracts visitors from all parts of the world. One of the largest single operations in the history of the reclamation of these fenlands, or as they are known locally "marshlands," was the construction of the Eau Brink Cut near King's Lynn. Prior to 1795 the River Ouse at this period was winding and slow running and consequently was continually silted up with sand shelves, only a very narrow channel being available both for navigation and drainage purposes, the former in those pre-railway days being of extreme importance.

A number of Acts of Parliament were passed between 1795 and 1831 to enable a new cut two-and-a-half miles in length and 300 to 500 ft. in width to be made to divert the river, the bed of the old river being filled up with the spoil from the new cut. The work was completed on June 21, 1821, at a cost of over half a million sterling. It is interesting to note that most of the work, employing large numbers of men, was executed during the Napoleonic wars, a striking commentary on the methods of making war in those days compared with the late war.

When the cut was made a bridge had to be erected as a channel of communication across the new cut, it being called the "Free Bridge," or "Cut Bridge." The first structure was opened in June, 1821, and was one of the longest wooden bridges in England, being upwards of 800 ft. in length. This bridge had an opening span in the middle, which, however, was practically never used after the coming of the railway era.

The cut and bridges were administered by the Eau Brink Commission until superseded by the Ouse Outfall Board in 1861 by Act of Parliament. A steel lattice girder bridge was built in 1873 from a design submitted in competition by Mr. C. H. Diver, and this structure, having shown excessive deflection, has for some years been restricted to light traffic. In September, 1923, the Norfolk County Council agreed to take it over and construct a new bridge (see pp. 12–13) in reinforced concrete. The engineer responsible for the design and construction is the County Surveyor of Norfolk, Mr. S. H. Warren, A.M.Inst.C.E.

The actual erection was commenced in August of last year. The new bridge will cost nearly £29,000. It consists of three spans of 120 ft. each and two spans of 69 ft. 6 in. each, and is of the arched rib type with suspended floor and tie. The piers and abutments are carried on piles, which are pitch-pine where totally submerged and reinforced concrete otherwise. Provision is made for a 20-ft. roadway and two 5-ft. footpaths. The roadway is carried by cross-beams which are cantilevered over to carry the footways. The ribs to the three main spans have a rise of 24 ft. and a cross-section of 4 ft. by 1 ft. 6 in. at the crown and 7 ft. 6 in. by 1 ft. 6 in. at the haunch. The smaller spans have ribs with a rise of 18 ft., the cross-section at the crown being 2 ft. 6 in. by 1 ft. 6 in. and 7 ft. 6 in. by 1 ft. 6 in. at the haunch. The total length of the bridge is 525 ft.

Concrete Cottage Competition.

THE result of the competition for designs for a concrete cottage, organised by Messrs. G. & T. Earle, Limited, of Wilmington, Hull, has now been announced, as follows: —

First premium (30 guineas)—Mr. Cecil A. Sharp, F.R.I.B.A., Westminster Chambers, 11 Victoria Street, Westminster, S.W.1.

Second premium (15 guineas)—Messrs. E. R. Hudson & Son, A.R.I.B.A., 69 Warwick Square, Westminster, S.W.1.

We hope to illustrate the premiated designs in our next issue.

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A RAILWAY GIRDER BRIDGE.

By S. A. BUNTING, M.A., M.Inst.C.E.

A BRIDGE had to be built recently to take the line of the Northern India Salt Department over the main line and sidings of the Bombay, Baroda, and Central India Railway (metre-gauge system) at Sambhar, Rajputana. The centre lines were at 45°, and the site did not admit of piers between all tracks. The minimum clear span (on the 45° skew) was thus fixed at 44 ft. The standard steel girders available were either 40 ft. or 60 ft., the first too short and the second too deep for the head-room required. Moreover, the cross-bracing connections were unsuitable for skew spans.

The author therefore designed a ferro-concrete structure, with three clear spans of 44 ft. (on the skew) over the main line and sidings and one of 23 ft. (on the square) over a well which could not be filled up. The depth of girder necessary admitted of the adoption of a deck type utilising the deck as main compression member, a more economical design than the through type for such a span. There was also room for 4½ in. of ballast below the sleepers, thus avoiding the direct fastening of the track to the bridge. Full allowance for impact, an addition of 50 per cent. to the live load, was insisted on by the railway company, just as it would be for a steel girder design with track attached.

There was doubt as to the four supports maintaining their relative levels when under rolling load. The girders are carried on rollers at the abutments, on tarred paper at one pier, and are fixed on the other pier, and this being so it was not worth while designing a continuous girder. The spans are also comparatively short. It was intended to carry all shear members over the piers from one span into the next, partly for convenience in anchorage and partly to obtain a slight reduction of the centre bending moments. This had been done in the construction of Gudha viaduct near by, where 34 spans of 24 ft. for a 2-ft.-gauge line were used, with no ill results. The railway company held that, not being designed as a continuous girder, the three spans should be in fact completely

A RAILWAY GIRDER BRIDGE IN INDIA.

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severed from each other. A compromise was arrived at ; no shear members cross the plane of division above the neutral axis, but the main tension members and shear members below the neutral axis were allowed to cross.

It is a difficult matter to crowd tension metal sufficient for a girder of this type into a web and yet keep the web thin and to keep the weight of concrete not carrying compressive stress low.

In a 60 ft. span double T-beam road bridge built by the author in 1912 four sets of ten eye-bars 4 in. by ½ in. F.B. (or nine 4 in. by ½ in. and two 4 in. by ¼ in. on the outsides) were used in each web, interleaved at the junctions and bolted through, the shear being taken by more flat bars bolted on to the tension members. There was room between diagonal shear members to core out six holes in each web 2 ft. 6 in. diameter, which lightened both the weight and the appearance of the girder. The thickness of web was 12 in., which held ten flat bars ½ in. thick, nine intervening ½ in. spaces and, theoretically, 1¼ in. cover of concrete on both sides.

Nothing of this kind was applicable in this case, the tension steel area required being greater while the railway regulations for shear necessitated reversed shear members in the central 11 ft. of the span. No reliance must be placed on the adhesion of concrete to the main tension members in such a case ; adhesion must be fully provided for by shear members (at 45°) attached to the main tension members, taking the whole of their tension by instalments up to the top flange and there distributing it by hooking the shear members over transverse deck members.

The railway company sent for comparison an English design similar to this for a 40 ft. clear span bridge for a broad-gauge (5 ft. 6 in.) railway. In this the adhesion difficulty was overcome by dividing up the tension area required into 15 round bars, bending these up as shear members into the top flange and anchoring them there one by one as the reduction in tension stresses toward the abutments allowed of it. But round bars of 50 ft. long are not available in places distant from centres of supply, and field-welding would have to be resorted to—too risky a matter for main tension members.

The original design was therefore adhered to, and rails used. Thirty-six foot rails were obtainable, which meant that all tension that could not be taken by the fishplates and fishbolts (in shear) had to be taken off by shear members to the top flange in the middle 36 ft. of the span ; this was quite possible, as it left only 4 ft. ($\frac{1}{11}$th of the span) at each end. Rail tension members have one great advantage in that holes can be put through the webs of rails for the attachment of shear members without reducing appreciably the tension section area. Flat-footed rails are not easy to protect by means of concrete adhering underneath, and a sound cover is very necessary to protect them from locomotive gases. This difficulty was dealt with by stretching ½-in.-mesh rabbit netting ½ in. below the rails and ½ in. above the moulds, the sides being turned up over the rail flanges. Liquid grout (1 : 1½) was run into the moulds till it showed above the rail flanges before ordinary 1 : 2 : 4 concrete was used at all.

The main tension members were two 60-lb. rails (6 sq. in. each) in each web, augmented in the centre of the span by three 2½ in. by ¾ in. flat bars on edge, with eyes at their ends, by means of which 2½ in. by ⅜ in. F.B. 45° shear members were bolted to each, 7 ft. from the span centre for one and 11 ft. from the span

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centre for the other two. These shear members were given a quarter twist and a 45° bend in the top flange, and extended 2 ft. 6 in. along the same.

All other shear and deck members were made of ¾-in. square bar, twisted, one complete turn in 6 in. length; ¾-in. square is readily obtainable in 17 ft. 8 in. or 18 ft. lengths. The twisting is done on a lathe or in a screwing machine at a cost of about 1*s*. per cwt., which is more than repaid in extra elastic limit strength in addition to the improved mechanical bond. If longer lengths than 18 ft. must be used and welding has to be resorted to, twisting is a very good test of the soundness of the weld; if no flaw can be found after twisting, the author is inclined to think that the weld can be absolutely trusted, though he has not had the opportunity of giving such welds a thorough tensile test.

All shear members were of U shape, passing through both rails at the base of the U, lying back at 45° in the web, and bent over two transverse deck members 9 in. apart, being hooked round the second. Their spacing varied from 2 ft. at the centre of the span to 9 in. over the piers.

Transverse deck members were spaced at 9-in. centres, the spacing being maintained by wiring on four ⅜-in. square longitudinals, one in each kerb and one just clear of each web. The thickness of deck and the fillets required for the distribution of main compressive stresses allowed of some sacrifice of transverse strength over the webs; the deck members did not have to lie at the top surface there. Thus three bends, one at each kerb and one in the centre, were sufficient for the deck members.

The ratio of the width of deck to the thickness and distance centre to centre of the webs being so much less than that usual in floor construction, the usual restriction as to the proportion of deck width to be reckoned as compression member was not valid; the whole deck (excluding the kerbs), the fillets, and the webs down to the neutral axis ($\frac{2}{3}$ the distance from deck to tension centre) were reckoned as compression area, with 600 lbs. per sq. in. stress at deck surface down to nil at the neutral axis.

Diaphragms 6 in. thick at the bottom and 7½ in. thick at the top (to ease removal of forms) with 6-in. fillets at deck junctions, were provided between the two webs at 7 ft. 4 in. centres. Over the piers practically the whole of the space between the webs was filled, more to ease formwork than to distribute pressure. Each diaphragm was reinforced with one ¾-in. tension member at the bottom and two ¾-in. cross-bracing members.

The two abutments were provided with rollers, groups of ten 2-in. round bars spaced by flat bars through which their reduced ends passed. The roller "paths" consisted of 12 in. by 4 in. channels, the upper channels embracing the girder ends and the lower channels embedded in the concrete abutment bedstones. Over the two piers lay the tension members, pairs of 60-lb. rails, 13 ft. 8¾ in. long, averaging 6 ft. 10⅜ in. each side of the centre lines of piers. Care had to be taken to wedge these rails off from the 36 ft. lengths as far as the fishbolts would let them go (about ⅜ in.), to ensure their taking their share of the tension.

One of the piers was a fixed support; vertical rods projected from the concrete bed stone to ensure this. The other pier was provided with a tarred paper slide, rollers being scarcely worth while for a 44 ft. span. The tarred paper started 1 in. thick, but was compressed to half that thickness as soon as the weight

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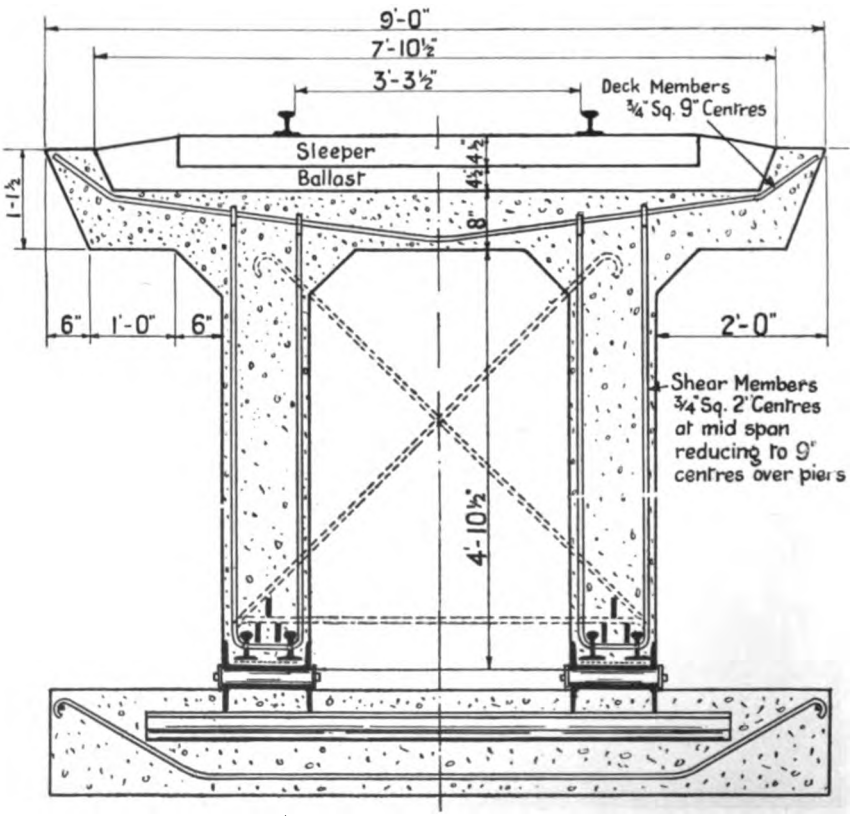
of the concrete came on it, a certain amount of tar being squeezed out. Tarred paper would have done perfectly well for the abutments also, distributing compression stresses evenly and deadening impact vibrations; it would connote semi-articulation over the piers, as it is impossible to foresee exactly how much the tarred paper would compress.

The forms were supported on trestles and contained by trapezoidal frames every 3 ft. 8 in. The forms for the inner sides of the webs and for the diaphragms consisted of boxes bottom uppermost, chamfered round the top edges to give the 6-in. fillet with the deck and tapered on all four sides to ensure easy removal downwards.

The top member of each trapezoidal frame had to be left unfixed till the main tension and shear reinforcement had been lowered into the webs. The top members were then fixed, strutted at the extremities by light scaffold poles, and packed up in the centre from the bottom members by struts inside the boxes and 2-in. square pegs on top of them, through the deck. Metre-gauge 41¼ lbs. track was then spiked on these top members.

No concrete mixing or chuting plant was available. The concrete materials,



CROSS SECTION OF GIRDER 44'-0" CLEAR SPAN
FOR METRE GAUGE RAILWAY.

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1 ft. of ¾-in. ballast, 6 in. of sand, and 3 in. of cement, were stacked dry in 10-ton falling-side trucks, half the floor of each truck being left clear for mixing. Trucks were shunted on to the bridge three at a time, their contents rapidly mixed and shovelled into the forms, and the empty trucks withdrawn to a dead-end siding on the embankment to make room for loaded ones. In the last span cast the track was continuous over the bridge, and back shunting was avoided. The 800 cubic feet were thus put in position in 2½ hours. A mixer could easily do it if the feeding arrangements were good, but quite an elaborate chuting plant would have been necessary to equal the speed of the trucks, and even then the work would have been more spasmodic. Pace is much more essential in a dry hot climate than in England. In India the stone, sand, and cement may have a temperature of 150° F. and the water 100° F.; British Standard Specification cement, good for two hours in this country, is often unsafe to handle there 20 minutes after wetting.

The concrete was mixed very wet to start with, proper penetration to all parts of the mould being more important than strength below the neutral axis. It was put in more and more dry towards the deck surface, any surplus water or slurry showing itself at the deck surface being promptly removed and replaced by good concrete before screeding. As soon as the surface could bear it, 3 in. of sand was spread over it and kept wet for ten days at least; this is better than sacks, which dry up at once in such a climate.

All four spans came out of the forms in perfect order and no cracks have developed in service. A 60-lb. track was laid on hard wood sleepers; stone ballast in all sizes from ½-in. to dust was used; the track was packed and boxed and a cement grout sprinkled over the surface to prevent the smaller material from being dislodged. The 2-in. square pegs already mentioned dried out and were removed, affording drainage holes every 3 ft. 8 in. along the centre line. This 60 lb. track, comparatively heavy for the service required, plays no inconsiderable part in distributing pressures. If rails could be obtained in the exact lengths from centre to centre of piers, and so laid, a further valuable reduction of impact stresses would be obtained.

A month after completion of the last span two metre-gauge "F" class locomotives and a train of ballast trucks loaded with sand to give axle loads of ten tons passed over at five miles per hour. Deflections taken by levels were too small to read, in any case considerably less than ⅛-in. The bridge was opened to traffic with a speed restriction of 5 miles per hour.

A more accurate test was made five months later. Long battens were fixed to the centre diaphragm of each span holding pencils whose points pressed lightly against cards pinned to stakes driven into the ground. A train drawn by two "F" class locomotives passed at 15 miles per hour, when no deflection exceeded $\frac{1}{16}$ in. and hardly more than $\frac{1}{32}$ in. In such a case, especially with tarred paper supports, simultaneous readings should be taken at the ends and in the centre. As the deflection read was less than half the calculated deflection, there was no need to pursue the matter further.

It is obvious that the bridge does not take up impact vibrations like a steel bridge, but whether the extra dead weight or the cushion of ballast is the chief factor in obtaining this result it is impossible to say.

A second bridge, also with three spans of 44 ft. and one of 23 ft., of the

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same type but of smaller dimensions, was built for the 2-ft.-gauge line parallel to the first bridge. The concreting was rapidly done by chuting from trucks on the metre-gauge bridge. To save time some of the formwork was done with rails, but two of the spans were moulded with the same forms, cut down, that were used in the metre-gauge bridge. The greater part of the wood purchased was thus used five times. The unit cost would be much further reduced if more spans had to be made.

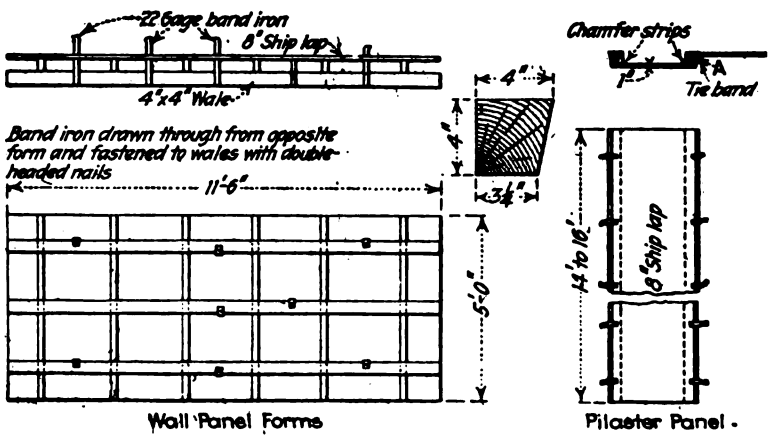
There is another field for ferro-concrete bridge work of this type, in the reclamation of old but sound steel girder bridges condemned as insufficiently strong to carry modern loads. These could be boxed in by forms, extra tension and shear members laid alongside the existing steel members, and a ferro-concrete deck provided to take the extra compression stresses. While such work could not be done under traffic, a week's interruption would suffice with Portland cement and less with a quick-setting cement. The increased loads, of course, would not be applied till the expiry of the full setting period.

RE-USING FORMWORK.

In building a reinforced-concrete structure at Appleton, Wis., for the Riverside Fibre and Paper Co., states *Engineering News-Record*, rather unusual wall and pilaster forms were used, wherein maximum re-use was had and excessive breakage avoided. Separate inner and outer pilaster forms were built of panel type. They were from 14 to 16 ft. in height, and special care was taken in aligning, plumbing and bracing them.

Intermediate wall forms were then built up in panels to a 6-ft. height and set in place between pilasters. These forms were held by a $\frac{7}{8}$ in. by 3 in. strip cleated to the mitred side of the pilaster forms. A 1-in. band of hoop iron was used instead of the usual wire tie.

After pouring, the wall panel forms were loosened and raised, the pilaster forms remaining in place. Panels were stripped by cutting the several bands and removing the strip, the mitred side of the pilaster form allowing the necessary clearance. A short 4-in. by 4-in. gin-pole was used from the scaffold above to raise the panels. All panels were numbered for position. By frequent oiling very little breakage resulted. Panels were used up to seven times without repair.



Details of Pilaster and Wall Forms.

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"THE CONCRETE YEAR BOOK, 1925."

THE second edition of "The Concrete Year Book" is now ready, and may be ordered by filling in the form on page xxxiv of this issue.

The first edition of this annual, issued on January 1 last year, was accorded an excellent reception by the press, and we have received many letters from purchasers expressing their appreciation, but perhaps the best indication of the value of the volume is the fact that the whole of the first edition, of 3,000 copies, was exhausted before the end of the year.

The new edition, which is edited by Dr. Oscar Faber, O.B.E., D.Sc., M.Inst.C.E., and the Editor of Concrete Publications, Limited, is on substantially the same lines as last year, that is, divided into three sections containing respectively a Handbook, a Directory, and a Catalogue.

A certain amount of the Handbook Section is retained, after thorough revision, and many new chapters have been added. It is the intention to give in this section the results of all important researches carried out during the previous year so that they will be available to subscribers in handy form and also to give authoritative articles on difficult aspects of concrete and reinforced concrete work and the latest developments in the manufacture and use of materials and in concrete construction. The chapters include "Cement"—in which rapid-hardening Portland cement, aluminous cement, coloured cement, hot cement, the storage of cement, etc., are dealt with; "Essential Factors in Making Good Concrete"—including tests for cement, quality of aggregates, water content, workmanship, setting time, density, rodding, the slump test, etc.; "Quantity of Water required for producing the Best Concrete"; "Quality of Water for Concrete"; "Tables for Reinforced Concrete Design"; "Concrete Floors"—in which designs are given for different types of concrete floors; "Concrete Sewers and Conduits"—in which every type of concrete sewer is illustrated and described; "Concrete Roads"—containing a specification for all-concrete roads, with notes on cost, materials, etc.; "Concrete Cottages"—giving elevations, plans and sections of a concrete cottage with a full bill of quantities; "Surface Treatment of Concrete"—with notes on exposed aggregate, coloured cement, colouring materials, rendering, American methods, cement washes, rubbed finishes, tooled finishes, etc.; "Fire Resistance of Various Aggregates." There are also chapters containing a great deal of memoranda of every-day use to designers and contractors, and a complete bibliography of every book published in the English language dealing with concrete, reinforced concrete, and allied subjects.

This year the Directory Section is much larger, and contains several new features, including a complete list of special trade names and brands in use in the concrete industry with the names and addresses of their proprietors.

The important Catalogue Section, which is complementary to the Directory Section, is also much larger this year. Its object is to give firms in the concrete industry, or catering for the concrete industry, an opportunity of describing their businesses or products in more detail than is possible in the Directory, and within the 180 pages of this section a very large number of machines, products, and businesses are described, with copious illustrations; this section should be invaluable to those seeking firms of contractors to undertake special kinds of work or a machine or product for a special purpose.

The book comprises 400 pages, and is sold at the low price of 2*s.* 6*d.* net.

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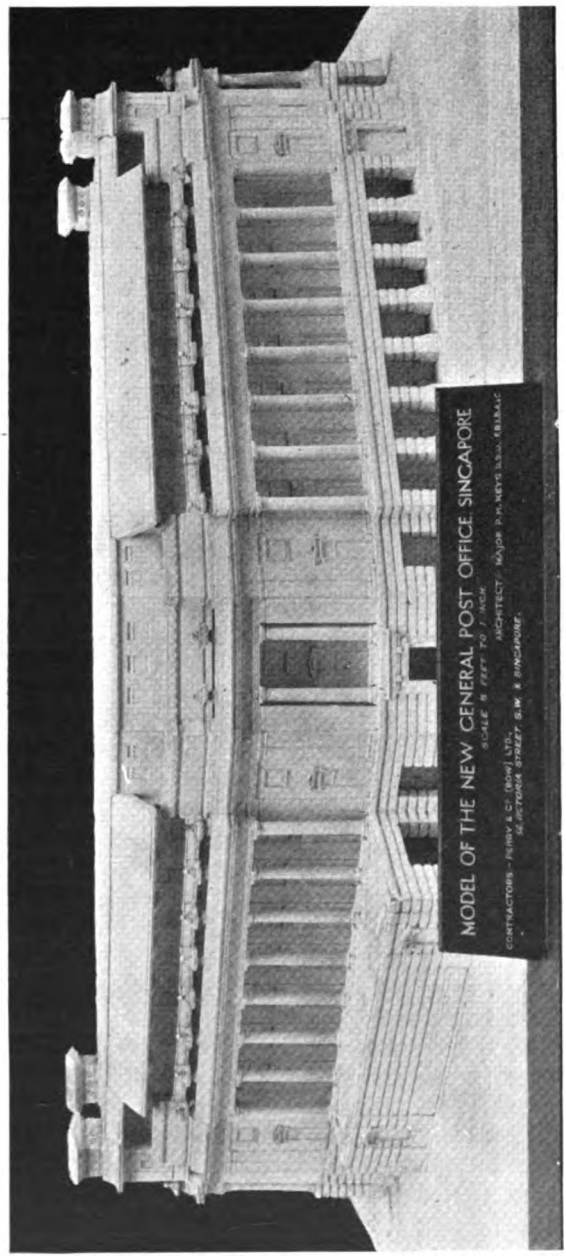

[*Major P. H. Keys, D.S.O., F.R.I.B.A., Architect.*

NEW GENERAL POST OFFICE BUILDING, SINGAPORE. (*See p.* 23.)

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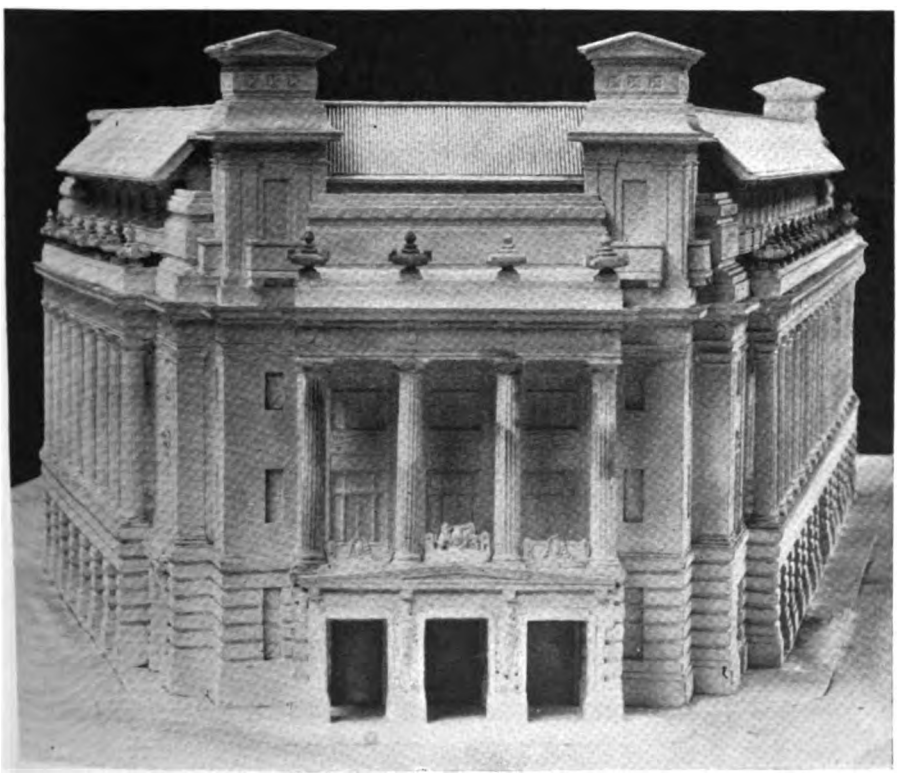
NEW GENERAL POST OFFICE BUILDING AT SINGAPORE.

We reproduce in this issue some photographs which have been taken of a plaster model which was recently exhibited in the Malaya Department at Wembley Exhibition, and which attracted a considerable amount of attention. This model is made to a scale of 8 feet to an inch and represents the new General Post Office Building which is now being constructed entirely in concrete and reinforced concrete at Singapore, Straits Settlements. The architect for the building is Major P. H. Keys, D.S.O., F.R.I.B.A., and the contractors in charge of the work are Messrs. Perry & Co. (Overseas), Ltd., of London and Singapore. The reinforced concrete plans are being prepared by Messrs. Edmond Coignet, Ltd., reinforced concrete engineers, of 125 Gower Street, London, W.C.1.

This building, when finished, will not only contain the new General Post Office, but also the Singapore Club and numerous offices for business purposes. The present site is occupied at one corner by the Singapore Club, which will be demolished when the new building is sufficiently completed for the Club to be transferred to the new premises.

The site of the structure is in a prominent position. The north-west elevation faces the river, and the two other principal elevations overlook two of the principal streets of the town, one of which is Fullerton Road. The site covers an area of approximately two acres. The principal façade will be 446 ft. long, and the maximum width of the building is



[*Major P. H. Keys, D.S.O., F.R.I.B.A., Architect.*

New General Post Office Building at Singapore.

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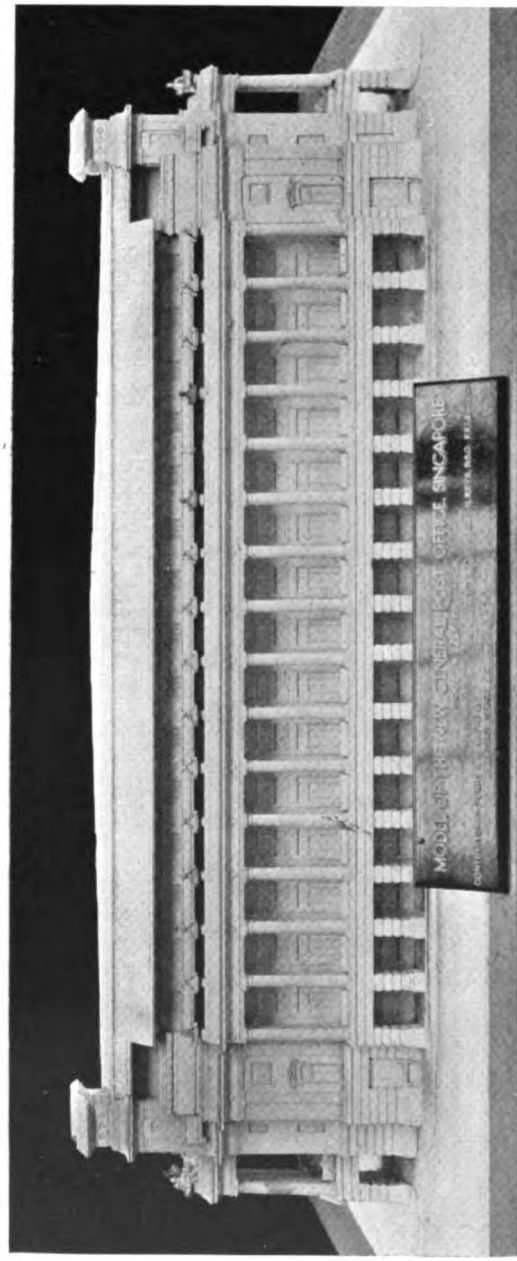

NEW GENERAL POST OFFICE BUILDING, SINGAPORE. [*Major P. H. Keys, D.S.O., F.R.I.B.A., Architect.*

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about 204 ft. Roughly speaking, the building is more or less triangular in shape. The total height of the structure measured from the level of the reinforced concrete raft to the top of the roof is 138 ft. There will be a basement, ground floor, and six other floors, with a flat roof. Owing to the uncertain nature of the ground, the foundations are composed of a cellular raft built in reinforced concrete and having a total height of 5 ft. 7 in. This raft has been designed to spread the weight of the building on to the ground so as not to exceed 1 ton per foot super.

The architectural features of the building, as far as the external work is concerned, may be appreciated by the photographs which we reproduce. The internal accommodation and general decoration will be carried out on the same scale, so that undoubtedly this building, which is estimated to cost about one million pounds, when completed will be one of the finest buildings in the East. All the external facings are to be carried out in a special artificial stone made with granite chippings and white cement, so that the general appearance of the building will be white. The building operations, which will take several years, will occupy a large number of native workmen under European supervision.

CONCRETE IN SEA WATER.

A FOURTH interim report has now been issued by the Committee of the Institution of Civil Engineers which is considering the deterioration of structures in sea water (London: Dept. of Scientific and Industrial Research, price 3*s.* 6*d.* net). This report, consisting of 73 pages and 12 plates, contains further reports on the specimens of timber, metal and concrete which have been exposed to the action of sea water, and many valuable reports of researches carried out in different laboratories to find the best methods of preventing the attack of marine borers. Most of the reports are concerned with the protection of timber, but Mr. L. H. Savile, M.Inst.C.E., Chief Engineer to the Admiralty, gives a brief but interesting report on defects in reinforced concrete structures at Admiralty establishments.

In consequence of a defect which was observed at a reinforced concrete structure, a thorough investigation of all similar structures at Admiralty establishments has been made, and it has been ascertained that, in some cases, deterioration has taken place.

This deterioration is attributed to the following causes:—

(*a*) Damage caused by collisions, etc.
(*b*) Insufficient "cover" allowed for in the original design.
(*c*) Insufficient "cover" through steel being displaced and forced near the surface when concrete was deposited.
(*d*) Concrete around steel being porous.
(*e*) Difficulty in properly surrounding certain types of shear members.
(*f*) Difficulty in properly packing concrete owing to the over-crowding of steel members.
(*g*) Bad junction between work done on different days, i.e., the surface of the concrete was not properly cleaned and roughened.
(*h*) Salt water used for mixing the concrete.
(*i*) Stray electric currents.
(*j*) Casings not having been properly cleaned out—nails, bars, shavings, wood blocks, etc., left in.
(*k*) Bad workmanship.
(*l*) Use of unsuitable sections of steel, causing difficulty in placing the links.

The remedial measures taken to prevent the occurrence of defects in future are as follows:—

(1) A greater thickness of cover than had hitherto been adopted, a minimum of 2½ inches in the case of principal members being arranged for.
(2) The proportioning of concrete in order to obtain maximum density.
(3) The size of aggregate to be limited to stones which will pass a ¾-inch ring.
(4) The use of flat steel stirrups or links to be prohibited, and overcrowding of reinforcement avoided.
(5) The coating of the surface of the concrete to prevent percolation of air or water, etc.
(6) Prohibition of salt water for mixing the concrete.

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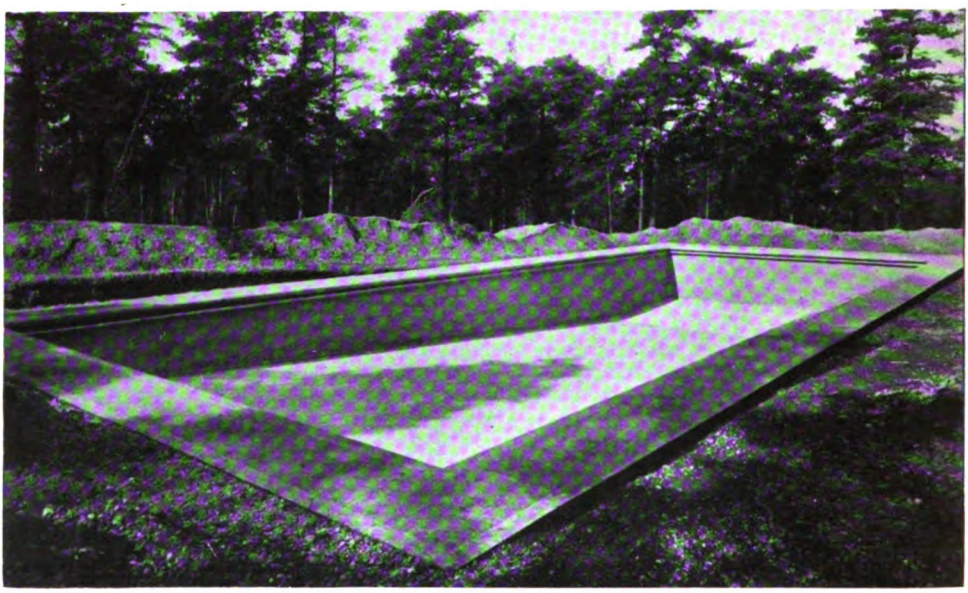

FIG. 2.—IMMEDIATELY AFTER COMPLETION.

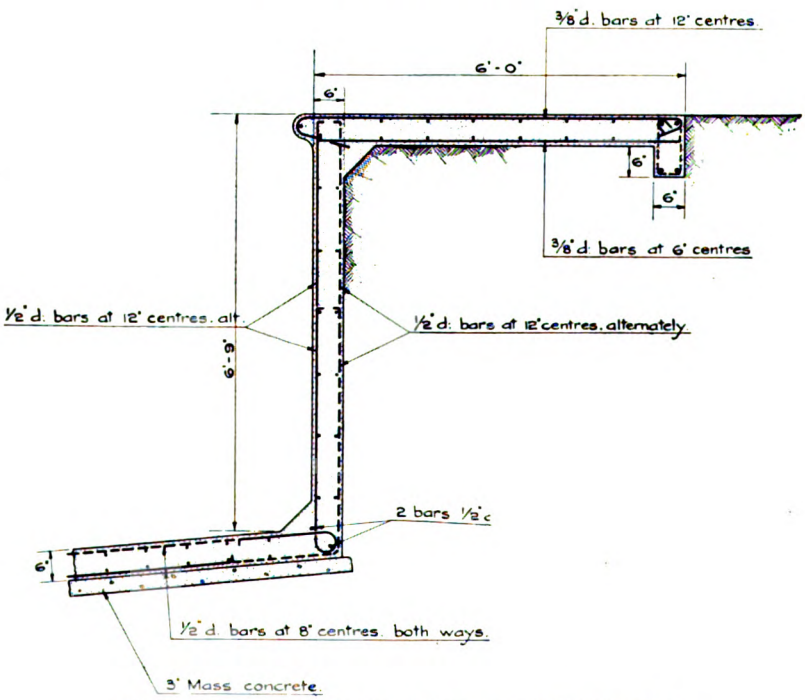

FIG. 1.—PART SECTION: SHOWING ARRANGEMENT OF REINFORCEMENT.

CONCRETE SWIMMING POOL AT ADDINGTON, SURREY. (*See p.* 27.)

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OPEN-AIR SWIMMING POOLS AT ADDINGTON AND BISHOP'S STORTFORD.

By W. L. SCOTT, A.M.Inst.C.E.

THE open-air swimming pools at Addington, Surrey, and Bishop's Stortford, Essex, giving practically the same accommodation and presenting similar appearances to the bathers, have recently been opened for use. The length and width of such swimming baths and pools are more or less fixed, for the reason that water polo and other aquatic sports render this desirable.

The bath at Addington is situated in the picturesque grounds of the Warehousemen, Clerks' & Drapers' Schools to which it belongs, and is provided for the use of the pupils attending this school. As an engineering structure it is without any particular interest, excepting perhaps that it is water-tight. Ordinary conditions of excavation being met with, the only provisions requiring to be made in the design were as follows:

(1) Against shrinkage of the concrete bottom and walls during the setting and hardening of the concrete.

(2) Against vertical bending in the walls and bottom slab due to lateral earth pressure when the bath is empty.

(3) Against horizontal bending at the junctions of the walls due to restraint of the cantilever walls at these points.

No question of foundation pressure arises in cases such as this, since the total weight of the materials and water introduced is less than that of the material excavated.

Fig. 1 shows a part section of the work and the arrangement of reinforcement employed. No asphalte coating or other expedient was employed to assist the concrete in retaining the water, concrete of the mixture used (1 : 1½ : 3 approximately) being quite impermeable to water provided the mixture is suitable to the component parts, which in turn must of course be of satisfactory quality.

Swimming baths of these types, being straightforward to construct, can be completed quickly and kept moist by filling the bath. Trouble from shrinkage during the hardening of the concrete is thereby eliminated. Fig. 2 shows the Addington bath immediately after completion.

The Bishop's Stortford swimming pool,

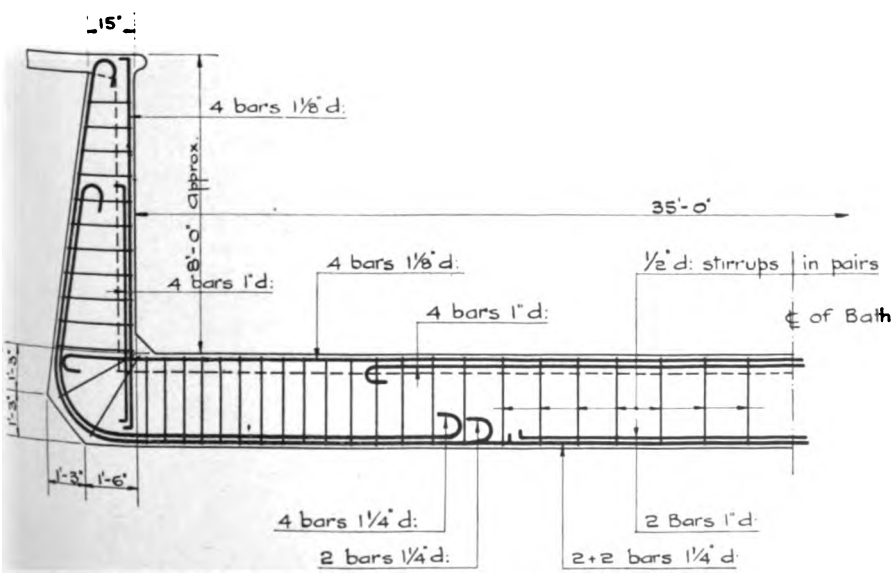

FIG. 3—CONCRETE SWIMMING POOL AT BISHOP'S STORTFORD: CROSS SECTION.

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although, as already stated, apparently a repetition of the one at Addington, in actual fact was a much more serious proposition from the engineering point of view. The nature of the ground and the presence of water at a level of 3 ft. 6 in. below the ground level at once introduced trouble. In the case of a bath subjected only to pressure from the contained water and an insignificant lateral wall pressure from the retained earth, no bending moments of any consequence are introduced, since the pressure from the contained water is carried directly by the earth below it without causing appreciable deformation to the bottom slab.

In the case of external water being present it will be seen that when the bath is empty a head of water gives an upward pressure upon the bath floor which, except for the weight of the floor, may be entirely unbalanced. Although it is not ordinarily possible to determine to what extent the pressure is present and causing upward bending of the floor between the walls, it is necessary to presume this to be hydrostatic, the head of water being dependent upon the maximum level to which the water present is observed to rise.

This being the case, it is necessary to design the bath bottom as a floor but with the customary conditions of loading reversed. An upward uniformly distributed load is therefore considered to be applied over the whole area of the bath floor, and this for the Bishop's Stortford Bath amounted to an average of 200 lbs. per square foot.

It is admittedly difficult to appreciate how such a total pressure as would be given on the floor can ever be realised in the circumstances usually obtaining in such cases, nor is it quite clear how such an upward pressure is taken by the weight and indeterminate frictional resistance of the side walls. The precaution of designing against such pressure, however, is justified as a result of the behaviour of those cases, not infrequent in the past, where a large area of the retaining medium has burst upwards or inwards due to such precaution not being taken in design.

Fig. 3 shows a cross section of Bishop's Stortford bath with the transverse beam carried up the walls in the form of vertical counterforts, which were provided to take the upward water pressure from the floor being transmitted to them by the bottom slab. *Fig.* 4

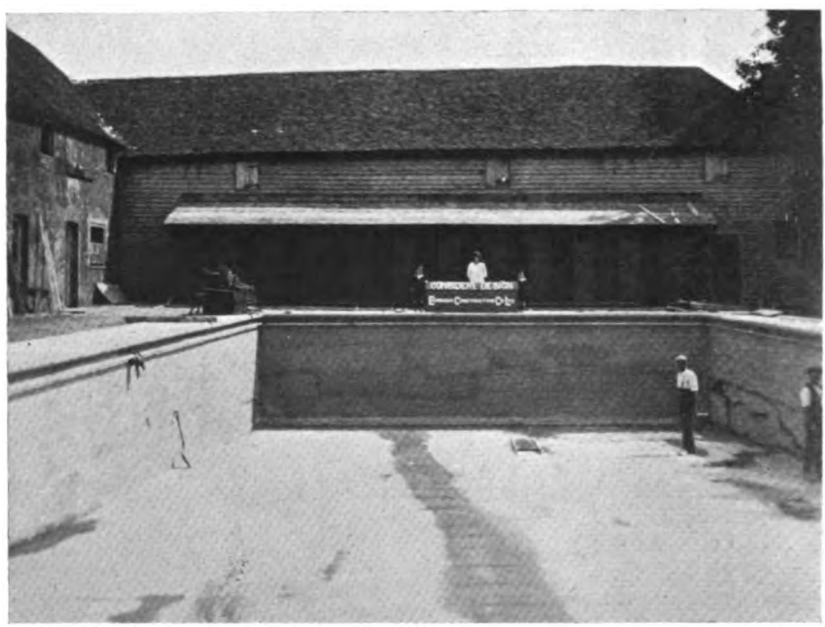

FIG. 4.—CONCRETE SWIMMING POOL AT BISHOP'S STORTFORD: IMMEDIATELY AFTER COMPLETION.

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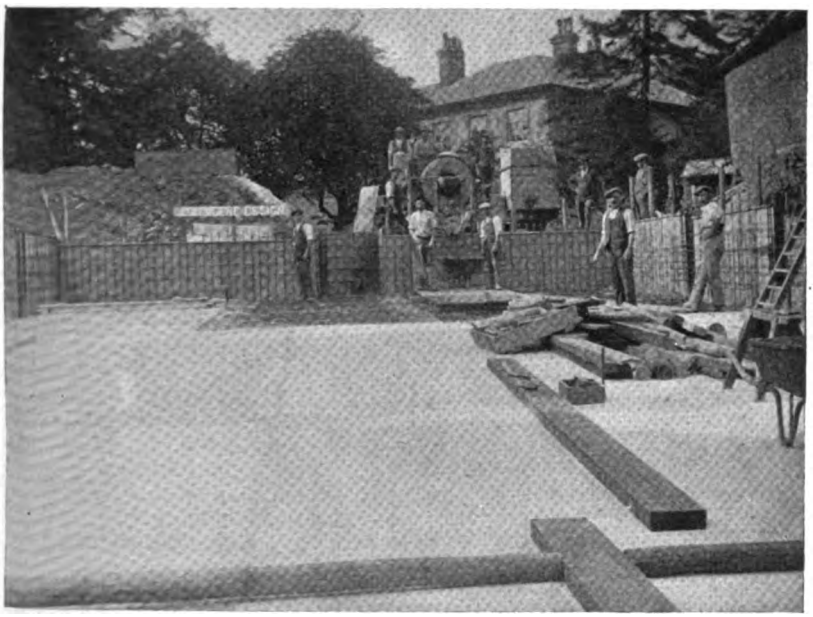

FIG. 5.—CONCRETE SWIMMING POOL AT BISHOP'S STORTFORD: DURING CONSTRUCTION.

shows a view of the finished bath and *Fig.* 5 the work in progress.

Sir Aston Webb & Son were the architects for the Addington bath, and Mr. H. L. Cabuche, O.B.E., F.S.Arc., the architect for the Bishop's Stortford bath, whilst Messrs. Considère Constructions, Ltd., were the designing engineers for both. The contractors for the two baths were Messrs. Edwards Construction Co., Ltd., High Street, Eltham, to whom every credit is due for the expeditious and satisfactory carrying out of both contracts.

Graphical Methods of Solving Continuous Beams.

IN the article by Mr. F. W. Davey, on "Graphical Methods of Solving Continuous Beams," in our last issue, the formula on page 754 should have read $\frac{S'' \times 2EI}{L^2}$ instead of $\frac{S'' \times 2EI}{L}$.

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QUANTITY OF WATER IN CONCRETE.

In our editorial pages we refer to the importance of strictly controlling the amount of water in concrete if best results are to be obtained, and we give below the rules adopted by Dr. Oscar Faber, O.B.E., D.Sc., M.Inst.C.E., for this purpose :—

For calculating the water required, take 28 per cent. by weight of the cement, and add 4 per cent. by weight of the sand and ballast. This gives the weight of water required to give a good working mix when the sand and ballast are dry and are of non-absorbent materials. It is applicable to any sand and any aggregate with the above qualifications, and therefore enables any particular case to be calculated in a very simple manner. In the case of absorbent materials, such as broken brick, the absorption of the brick should be calculated and the additional water required to soak the brick thoroughly should be added. If, for example, a certain concrete contains 8 cu. ft. of aggregate and requires 10 gallons of water per bag of cement when the aggregate is non-absorbent, and it is then desired to substitute broken brick for the aggregate, an experiment should be made to ascertain how many gallons of water are required to fill the pores of 8 cu. ft. of broken brick. Supposing this is found to be 2 gallons, then the quantity of water would require to be increased from 10 gallons to 12 gallons. The easiest way of ascertaining this additional quantity of water is to weigh 8 cu. ft. of ballast in the dry state, soak it in water, and weigh again. The difference in weight represents the water absorbed, and, remembering that 10 lb. of water go to 1 gallon, the result is easily ascertained.

This formula is based on sand and ballast both being dry. In practice this is not always realised, as the materials are generally left in the open and may be subject to showers of rain, or may have been dredged from under water, or washed. Where the materials themselves contain a certain amount of water, the quantity of added water must of course be reduced. Perhaps the best method in practice of controlling this on a job is artificially to dry a sufficient quantity of sand and ballast over a stove and then make a trial mixture of concrete with the correct quantity of water given by the formula, carefully observing the stiffness or plasticity of the resultant concrete. A simple test on the wet materials can then be made to see what quantity of added water is necessary to give the same plasticity.

The table shows the water required for different concretes, expressed either per bag of cement in gallons, or per cu. yd. of sand and ballast for various concrete

Quantity of Water to give Best Results for Reinforced Concrete with Dry Non-Absorbent Materials

Mix.	Weight of Materials. Lb. per bag of cement.			Volume of Materials. Cu. ft. per bag of cement.			Lb. per bag of cement.			Water per bag of cement. Galls.	Water per cu. yd. of sand and ballast
	Cement.	Sand.	Ballast.	Cement.	Sand.	Ballast.	28% Cement.	4% Sand and Ballast	Total.		
1	2	3	4	5	6	7	8	9	10	11	12
1, 4, 8	200	800	1·600	2·22	8·88	17·77	56	96	152	15·2	15·4
1, 3, 6	200	600	1·200	2·22	6·66	13·33	56	72	128	12·8	17·3
1, 2, 4	200	400	800	2·22	4·44	8·88	56	48	104	10·4	21·1
1, 1½, 3	200	300	600	2·22	3·33	6·66	56	36	92	9·2	24·9
1, 1, 2	200	200	400	2·22	2·22	4·44	56	24	80	8·0	32·4

NOTES.

(1) With absorbent materials, add enough water to provide for full absorption.
(2) When materials are wet, reduce water, but not below that quantity given in Column 8.
(3) Cement, sand, and ballast are taken as weighing 90 lb. per cu. ft.
(4) Cement should always be measured by weight.

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mixtures. This table is for non-absorbent and dry materials, and adjustment must be made when these conditions do not occur. The table is based on the cement, sand and ballast weighing 90 lb. per cu. ft., which is generally approximately correct, though absorbent materials like broken brick generally weigh less.

In practice it is of course important that the cement should always be measured by weight, seeing that unreliable results are obtained when it is measured by volume owing to the great variation of density which can occur according to whether the cement has been thoroughly shaken up with air or has been thoroughly consolidated by vibration and pressure. In other words, while 1 cu. ft. measure can by vibration or pressure be made to contain 90 lb. of cement it is also possible to pour the cement loosely into this measure in such a way as to fill it completely with only 60 lb. of the same cement. This point does not need further emphasizing, and is best dealt with in practice by specifying so many cu. ft. of sand and so many cu. ft. of ballast to one bag of cement weighing 200 lb. It is for this reason that the quantities in the table are made on the basis of one bag of cement.

BOOK REVIEW.

Refractory Materials: Their Manufacture and Uses. By Alfred B. Searle.

London: Charles Griffin & Co. Ltd. Pp. 730+xv. Price 42s. net. Second Edition.

THIS is a book designed to be of service to both manufacturers and users of refractory materials, and in the latter category will be found the most important industries of the world. All commodities in the production of which heat plays a part are dependent to a greater or less extent upon refractories. The use of refractory materials is necessary for a vast number of purposes, ranging from kitchen fires to the enormous furnaces in which iron, glass, cement, etc., are produced.

The author deals exhaustively with the raw materials for the manufacture of firebricks, retorts, crucibles, and furnace linings, and then proceeds to describe in detail the processes adopted for the production of the numerous types of refractory materials. In addition to the ordinary fireclay and silica rocks, which are the typical British raw materials for refractory purposes, there are graphite, bauxite, chromite, zirconia, and many other minerals which are coming into use on account of their suitability for heat resistance under special conditions. Although the United Kingdom is rich in fireclays of good quality, the author asserts that British refractories are often inferior to the Continental article because the manufacturers of the latter have been quicker to adopt scientific methods of production, and in particular have a greater knowledge of the properties of their products, which enable them to satisfy purchasers of their suitability for the purposes required. The British refractory materials industry is well-endowed with suitable raw materials, but has been content to reply upon "rule-of-thumb" in the manufacturing processes, while the Continental industry has been able to exploit raw materials, often inferior to British, to better advantage by adoption of scientific methods. This unfavourable comparison from the British point of view is one that occurs too frequently, but could be corrected in the case of the refractories industry by taking advantage of the facilities that exist for co-operative scientific research.

The book deals fully with the application of refractories to the more important industries, such as steel, glass, cement, gas, etc., and the work may be recommended to the technical men in those industries as a useful source of information and reference. Books such as this which summarise information from scientific records in various languages, which would otherwise be inaccessible to many, are of great value, and when, as in this case, the author is an expert in the subject dealt with the reader is able to accept the records with confidence and can frequently apply the information with advantage to the industry in which he is interested.

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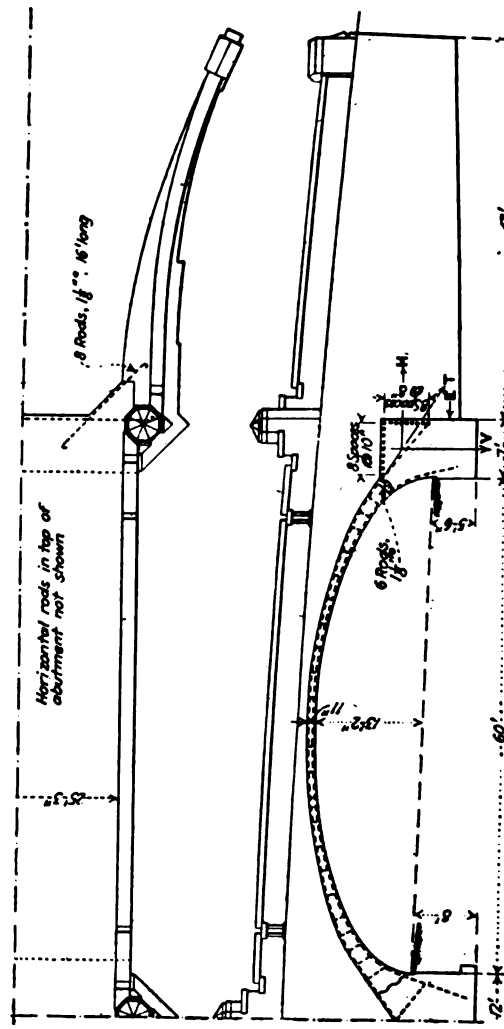

HALF PLAN AND ELEVATION, AND ALTERNATIVE DESIGN AT ABUTMENT.

REINFORCED CONCRETE BRIDGE AT YONKERS, NEW YORK. (*See p.* 33.)

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AN ECONOMICAL TYPE OF ABUTMENT FOR ARCH BRIDGES.

A TYPE of abutment for reinforced concrete arch bridges for which a considerable economy compared with other designs is claimed, is described by Mr. A. G. Hayden, Designing Engineer to the Bronx (N.Y.) Parkway Commission, in a recent issue of *Engineering News-Record*.

The bridge consists of two stone-face elliptical reinforced-concrete arches each 60 ft. clear span. The park-way drive passes under one arch, and the Bronx River will, when diverted, flow under the other arch.

The reinforced concrete arch ring, it is stated is 11 in. thick at the crown and is probably much lighter in section than is usual for like bridges subject to heavy traffic, except near the springing where maximum economy cannot be realised on account of the limitations imposed by the elliptical shape of the intrados. Further economy was realised by taking advantage of the long wing walls in the approaches, which were required for architectural appearance, to save in cost of abutments. The arch ring is designed to deliver the total thrust at the springline to the wing walls, at which point the thrust is resolved into horizontal and vertical components. The horizontal component is taken by the wing walls and dissipated into the ground foundation. The vertical component of the thrust if concentrated at the wing walls would result in excessive soil pressures at these points, so the abutments are made to act as vertical beams to distribute the vertical load nearly uniformly over the entire area of their bases.

Ordinarily this would be the only function to be performed by this type of abutment. In this particular case the wing walls are not in line with the edges of the arch ring but are offset at the pylons so that the arch ring does not butt directly into the walls. Thus the abutment must act also as a horizontal beam to transfer the horizontal thrust to the walls and must therefore be wider at the top than would otherwise be necessary.

Referring to *Fig.* 1, the horizontal lines of rods in the tops of the abutments resist tension due to the action of the beams in distributing the vertical components of the arch thrust and the vertical lines of rods in the backs of the abutments resist tension due to the action of the beams in transferring the horizontal components from the corners of the arch ring to the ring walls. If the arch ring butted directly against the wing walls the design of the abutments could be as shown in *Fig.* 2. A tentative design for abutments of the conventional gravity type for this bridge was also made and the comparison with the adopted design showed that the latter effected a saving in cost of excavation and concrete for the two abutments amounting to about £330. The scheme shown in *Fig.* 2, if it could have been used by avoiding the offset wing walls, would have permitted a further saving of about £260.

At first sight it might appear that the thrust for each foot width of arch should be resolved at the abutment into horizontal and vertical components, the vertical components going directly into the foundation and the horizontal components being carried by horizontal beam action of the abutments to the wing walls; but a little consideration will show that, on account of the extreme stiffness of the arch ring axially as compared with the stiffness of the horizontal beam, the conditions would not be as supposed. Without longitudinal slip-joints in the arch ring the abutments would not take off the thrust uniformly, but nearly the total thrust would be delivered to points near the wing walls even though the abutments were designed to function otherwise.

The effect of earth pressure behind the abutments was considered in the actual design, but the force due to this is not indicated in the figures.

The architectural design is by Messrs. Bowdoin and Webster, and the structural design by the Parkway Commission's designing engineers under direction of Mr. Hayden.

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DESIGN OF FORMWORK FOR REINFORCED CONCRETE CONSTRUCTION.

By A. E. WYNN, B.Sc., A.M.Am.Soc.C.E.

IV.—DESIGN TABLES (*Continued*)

Table 5—Ledgers and Wales

This Table gives the size and span of ledgers supporting joists, or wales supporting studs, for various spacing of joists and studs. They are assumed to be continuous over two spans or more.

The concentrated load brought by the joist or stud is first calculated and the nearest to this load is then found in the table, which will give the size and maximum span of the supporting member. This maximum span governs the spacing of the posts.

Generally, several alternate sizes and spans can be used, so it is necessary to make alternate designs to determine which is the most economical.

This Table is calculated from formula 25. The live load can be assumed at 75 or 40 lbs. per sq. ft. as desired; generally the latter is sufficient.

Shear and deflection can be investigated by formulæ 26 to 32. For shear one joist should be assumed to be close to the support.

The width of the member is also important, as the allowable bearing stress on the post should not be exceeded (see formula 23).

If the post is not square the long side should be placed parallel to the ledger.

Two small sizes are often doubled up to make a ledger or wale, using the same size as for joists and studs, thus keeping the number of different sizes to a minimum and also saving the extra cost of the larger sizes.

The ordinary sizes used are 3 in. by 4 in. or 6 in. to 4 in. by 4 in. or 6 in., either singly or by doubling up.

Tables 6 to 9—Column Yokes

These Tables give the required spacing of column yokes for various sizes of yokes and heights and dimensions of columns. They are calculated from formulæ 37 and 41. Strength in bending will govern the spacing; except where noted the deflection will be less than $\frac{1}{8}$ in. Shearing stress will be safe. It is assumed that the span of the yokes is 1 ft. greater than the dimension of the column.

For the short side of a rectangular column or the side of a square column that does not carry the bolts, the span will be equal to or less than the side of the column; if desired the yoke can therefore be made smaller, as noted, but the spacing will be the same.

It will be noted that in some cases two sets of spacing are given to a column; this is when the strength of the sheathing governs the spacing. Where only one set of spacings is given either of the thicknesses of the sheathing given, or greater, can be used.

Yokes are not often spaced closer than 9 in. at the bottom; when a

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closer spacing is required it is better to use a heavier yoke or to put a centre bolt through the column and so reduce the span. When a centre bolt is used the same spacing should be used as for a column, of half the dimension of the side.

Columns are poured to the underside of the deepest members framing into them, so that the height to take for the column will be the height from the bottom of the deepest beam to the floor or footing below.

For any height the first yoke from the bottom should be placed about 4 in. up, and the spacing above should start with the distance cut by a horizontal line through the height of the column; or, if this distance is cut at about the centre, start with the average between this and the distance next above. For instance, for a column 9 ft. high and 18 in. side, place the first yoke 4 in. up and the spacing above will be 15 in., 16 in., 18 in., etc., with 3 in. by 4 in. yokes.

It is common, though not good, practice to space the yokes equally all the way up at, say, 14 in. to 16 in. on centre for ordinary size columns and heights in buildings. If this spacing is suitable for the lower yokes it will mean only a waste of material above, but if a smaller spacing should be used at the bottom the lower yokes are liable to bulge, and this is often the case in practice when equal spacing is used.

It is often claimed that it is easier for a carpenter to lay out equal spaces. This may be so, though it is doubtful; the saving of a yoke per column will, however, more than offset any possible extra labour cost in laying out the yokes correctly.

When column forms are to be used over and over again on successive floors, with varying heights and sizes of column, the spacing of the yokes in the first place should be such that they are on the safe side for the worst conditions.

For column forms to be used only once or twice 1 in. sheathing is sufficient, but if the forms are to be used several times over it is better to use 1¼ in. sheathing. Sheathing thicker than 1¼ in. is seldom used except for unusual conditions.

The Tables are for dressed timber; for rough timber the spacings given may be multiplied by the same factors as given for joists, except where the spacing is governed by the safe span of the sheathing.

The sizes most used are 3 in. by 4 in. or 4 in. by 4 in.

Table 10—Wall Ties and Wales

The spacing of studs is governed by the maximum span of the sheathing from Table 2, to which may be added the width of the stud.

If the studs are simply tied through the wall with no wales, the Table will give the maximum spacing for the ties. If wales are used, as is customary, the Table gives the spacing of the wales. It is calculated from formulæ 49 and 50, the deflection governing the spacing and the pressure being assumed constant between any wale and the one above it, with intensity equal to that at the lower one.

When wales are used they need not be tied through the wall at every stud, but the maximum span between ties can be found from Table 5

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for the size it is desired to use. Care must be taken to see that all the studs have a bearing on the wale, wedging them if necessary when a stud is a little under size.

For any height of wall place the first wale or tie about 12 in. from the bottom and start the spacing from that point with the distance cut by a horizontal line through the height of the column.

Wall forms more than 12 ft. or 14 ft. high are seldom built, as it is usually cheaper to build the forms for half the height and raise them. If the forms are built to the top for high walls, they are generally poured in two lifts. In either of these cases, therefore, the spacing of the wales need only be that for a wall of half the total height. In some cases for strength and watertightness a high wall has to be poured without a break, in which case the forms must be designed accordingly.

It must be remembered that the strength of wall forms will depend on the tie wires or bolts, and the Tables will give the maximum spacing of these. They should not be used too sparingly ; additional ones should be added at weak points and corners, and outside braces also used.

For low walls, 2 in. by 4 in. studs and 3 in. or 4 in. by 4 in. wales can be used ; for walls over 10 ft., 3 in. or 4 in. by 4 in. studs and 4 in. or 6 in. by 6 in. wales may be used.

One-inch or 1¼-in. sheathing will do for any except heavy retaining walls.

Wales are often made by doubling up smaller sizes with the bolts placed between them.

Table 11—Posts

This table is calculated from formula 33. Posts over 8 ft. in height should be braced in both directions at their centre, usually with 1 in. by 6 in. timber, and the height then for calculating the allowable load will be half the total height of the post.

For very high posts the bracing should be at the third points.

If the bottom of the timber carried by the post is wider than the post, the size of the post is governed by the bearing stress on the timber, either 400 or 600 lbs. per sq. in. according to the quality of the timber or whether hardwood caps or steel bearing plates are used.

It will be seen that theoretically the loads depending upon bearing stress are equivalent to those for columns 10 ft. high and over. Nevertheless, bracing should always be used. The formula for posts assumes no eccentricity of loading and hence no bending, but in formwork some bending action will nearly always be present, chiefly because the posts are seldom placed absolutely vertical and often several inches out of plumb, in which case a much smaller load can safely be placed on the post. Bracing by reducing the unsupported height will take care of this bending action.

Wedges, preferably hardwood, should always be placed beneath the posts to allow for adjustment in height.

For rough posts, add 10 per cent. to all loads given in the Table.

For ordinary building work, 3 in. or 4 in. by 4 in. posts are used and,

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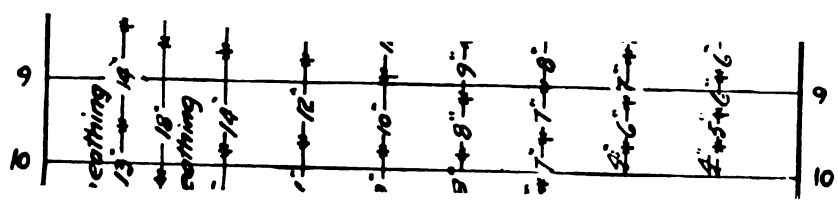
TABLE. 5.

MAXIMUM CONCENTRATED LOADS CARRIED BY LEDGERS AND WALES FOR VARIOUS SPANS AND SPACING OF LOADS

	18	250	450	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	21	290	530	840	460	830	1320	620	1130	1790	960	1740	2750
	24	330	600	950	520	950	1500	710	1290	2040	1090	1980	3130
	27	360	650	1020	560	1020	1610	760	1390	2190	1170	2130	3360
	30	390	700	1100	600	1100	1740	820	1500	2370	1270	2300	3620
	33	420	760	1200	660	1190	1890	900	1630	2570	1380	2410	3940
	36	460	830	1310	720	1310	2070	980	1780	2820	1510	2740	4320
	39	510	920	1450	800	1440	2290	1080	1970	3110	1670	3030	4770
	42	570	1030	1630	890	1620	2560	1210	2210	3480	1860	3380	5540
8'-0"	12	150	270	430	230	430	680	320	580	920	490	900	1420
	15	180	330	530	290	530	840	400	720	1140	610	1110	1740
	18	220	400	620	340	620	990	470	850	1340	720	1300	2060
	21	250	460	730	400	720	1150	540	990	1560	830	1510	2390
	24	300	550	860	470	860	1360	640	1170	1850	990	1800	2830
	27	320	580	920	500	920	1450	690	1250	1970	1060	1920	3030
	30	340	620	980	540	980	1560	740	1340	2110	1130	2050	3240
	33	370	670	1060	580	1060	1680	800	1440	2280	1220	2210	3490
	36	400	730	1150	630	1150	1820	860	1560	2470	1320	2400	3780
	39	440	800	1260	690	1250	1980	940	1700	2690	1440	2620	4120
	42	480	870	1380	760	1370	2180	1030	1870	2960	1580	2880	4540



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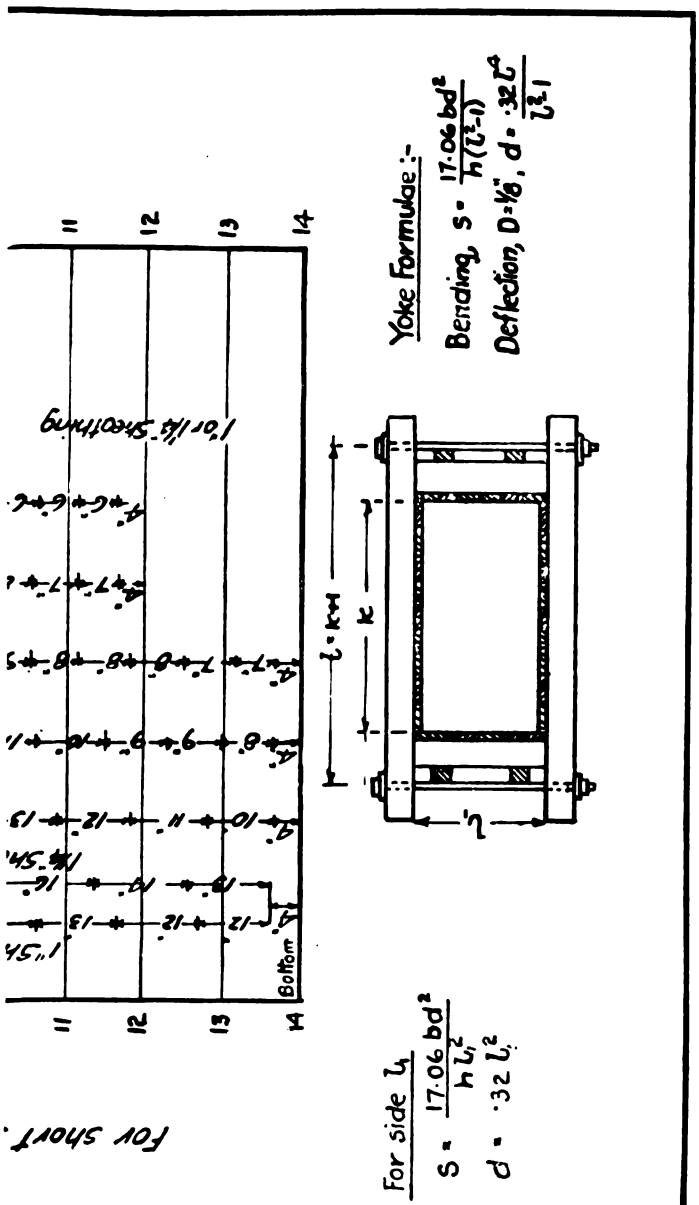
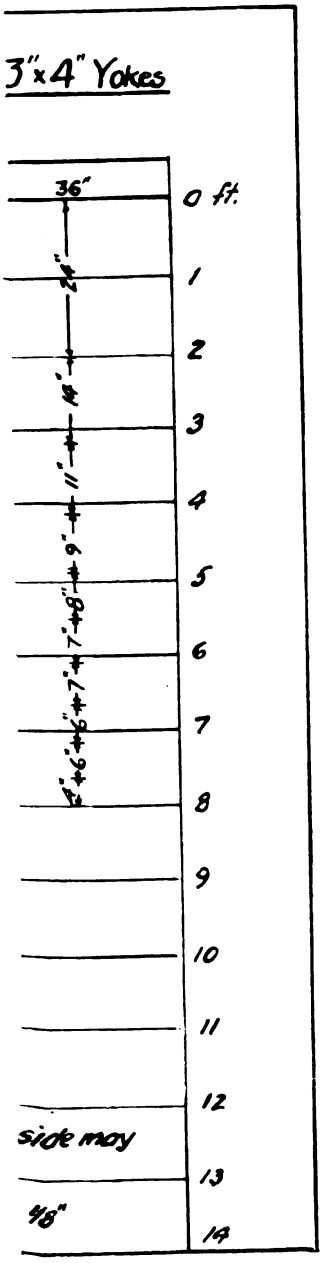
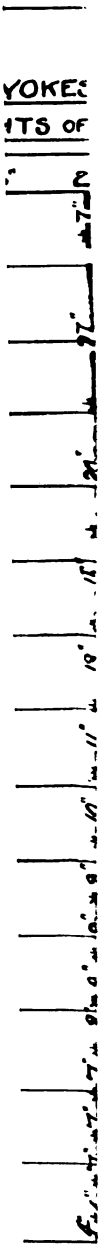


: SPACING FOR COLUMN YOKES.





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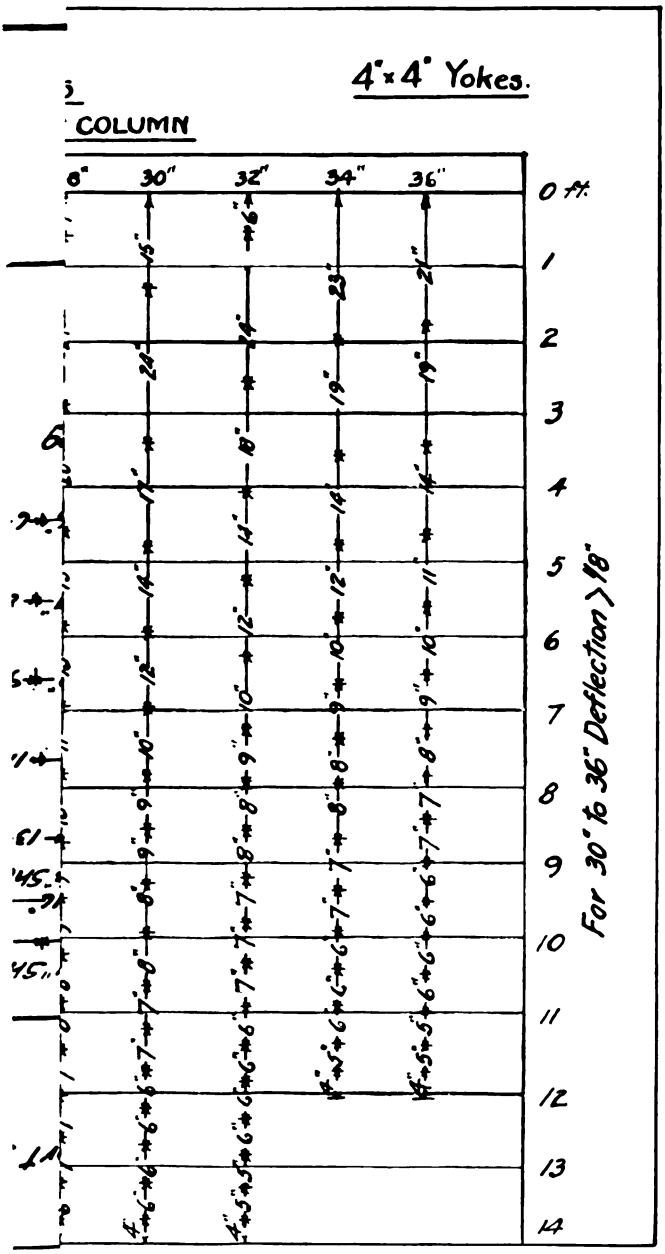

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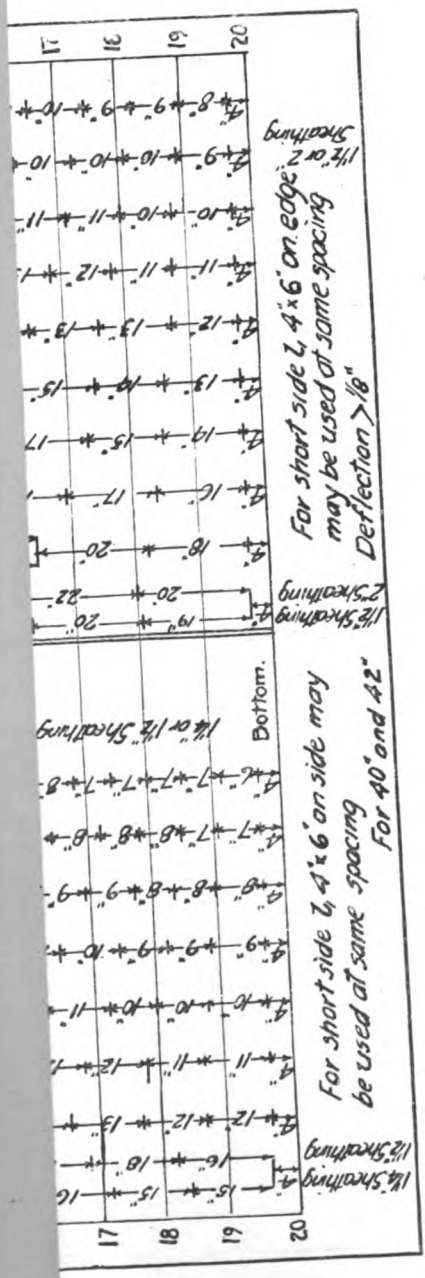

DESIGN OF FORMWORK FOR REINFORCED CONCRETE CONSTRUCTION.

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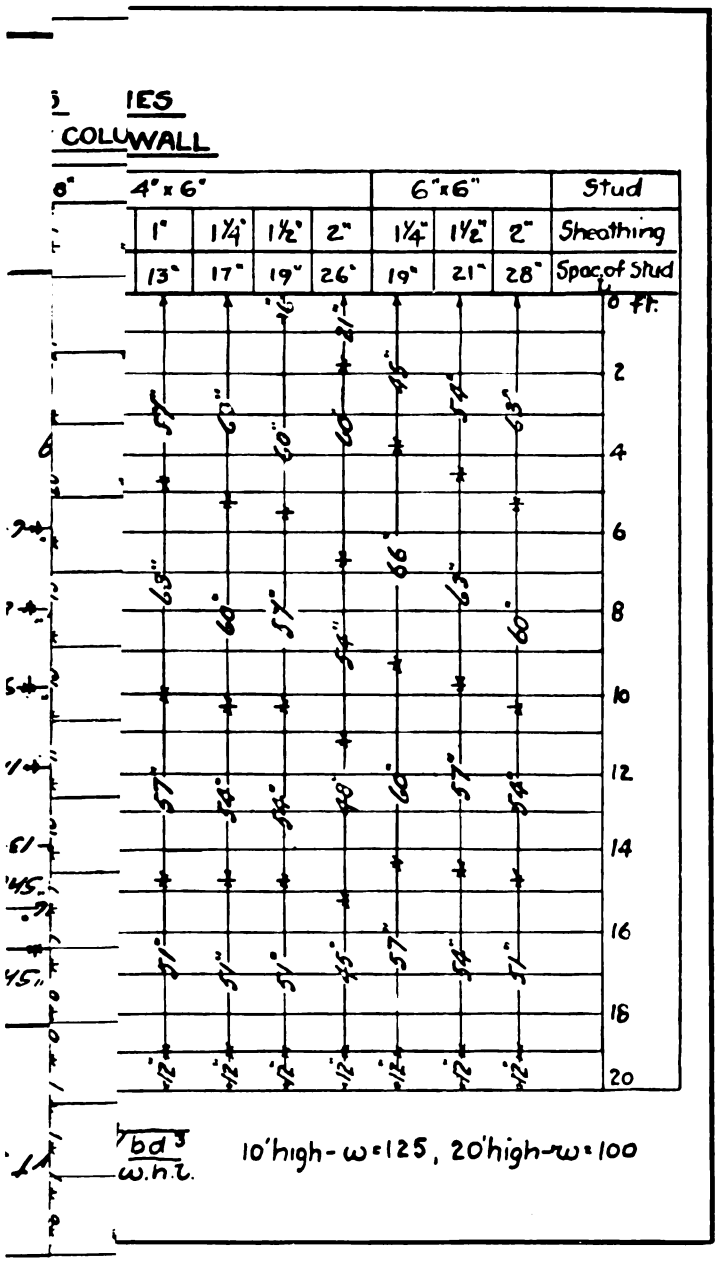

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TABLE. II.
SAFE LOADS ON POSTS

$W = 1000(1 - \frac{l}{80d})bd$

For Bearing:-

$W = 400.b.d.$

or $= 600\,b.d$

Size of Post	3"x4"	4"x4"	4"x6"	6"x6"
Bearing @ 400	4100 lbs.	3600 lbs.	8600 lbs.	13300 lbs.
" " 600	6200	8400	12900	20000
Height 4'	8000	11800	18100	29600
6'	7000	10700	16400	27900
8'	5800	9500	14600	26100
10'	4700	8400	12900	24400
12'	3600	7300	11200	22700
14'	2400	6200	9500	21000
16'	1300	5100	7800	19300
18'	180	3900	6000	17600
20'	—	2800	4300	15900

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for heavier work 6 in. by 6 in. It is a common rule amongst superintendents that a 3 in. or 4 in. by 4 in. post will carry a cu. yd. of concrete, or 4,000 lb.

Beam Bottoms

A table is not necessary for the design of beam bottoms, since the allowable span does not vary greatly with the size of the beam. They are designed as continuous shallow beams supporting a uniform load and spanning between the supporting posts. Formulæ 3 and 14 apply, "*w*" being the weight of the concrete beam and the live load on the beam. It is assumed that none of the slab load is carried by the beam bottom, and that the beam bottom is independent of, and therefore receives no support from, the side forms.

For a deflection of $\frac{1}{8}$ in., deflection will govern the allowable span between posts.

Two-inch dressed timber is almost invariably used for beam bottoms. The maximum span is independent of the width of the beam and varies only slightly with the depth, the depth being measured from the top of the slab.

With 2 in. dressed timber, a deflection of $\frac{1}{8}$ in., and a live load of 40 lbs. per sq. ft., the following rule can be used :—

For depth of beam up to 16 in.,		space posts 4 ft. 3 in.	apart.
,,	from 16 in. to 24 in.	,, 4 ft. 0 in.	,,
,,	,, 24 in. to 36 in.	,, 3 ft. 9 in.	,,
,,	,, 36 in. to 48 in.	,, 3 ft. 6 in.	,,

Posts supporting beam bottoms should always have a cross-piece at the top, braced back to the post, in order to give the bottom full bearing.

Application of Tables

We now have all the data necessary quickly to design any problem in formwork or to check the strength of formwork already erected.

Sizes should not be picked from the Tables at random without considering the two important points—timber available and relative cost of different sizes.

It is not economical to hold up work waiting for a certain size timber when another size that can be delivered immediately can be used equally as well.

Also, before deciding on a design the question of how many times over it will be necessary to use the forms must be considered. Light forms will not stand much re-handling, and in a multiple-story building it will pay to put more timber into the built-up units of the first floor than is required for strength, because less patching and renewals will be required on successive floors.

For almost any condition several different designs can be made, all having the same strength, and it is necessary to make comparisons of the relative economy unless there is no choice in the sizes of members to use.

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PERMEABILITY OF CONCRETE.

THE results of a series of tests on the permeability of broken-stone concrete, conducted by Professor Owen Withey and Mr. C. A. Wiepking at the University of Wisconsin Engineering Experiment Station, have been published as a Bulletin (No. 1245) of the University.

The test specimens were made of Portland cement assumed to weigh 100 lbs. per cubic foot loose, sand, and a soft magnesian limestone crushed and screened through a 1½-in. screen. Proportions of 1 : 1½ : 3 and 1 : 2 : 4 (by volume) and 1 : 9 (by weight) were selected for most of the mixes, because such concretes are in the range of proportions which are used with similar materials in practice, and also because the apparatus available for the tests would not detect leakage into concretes richer than 1 : 1½ : 3, and mixes leaner than 1 : 9 were not dense enough to hold a pressure of 40 lbs. per sq. in. in testing.

All the batches of concrete except five were made into the "PU" type of specimen. This type was developed after a number of other designs had been experimented upon, and had previously been used with success in a series of impermeability tests on gravel concrete. *Figs.* 1 and 2 show a sketch and a photograph of the "PU" type of specimen. The area of the concrete core under test was 1 square foot, so that the leakages were determined directly in terms of this unit of area. Since the results of tests on this type of specimen depend on the imperviousness of the mortar shell, it was important that a rich and properly mixed mortar be used, and that a proper bond be obtained between the mortar, the iron casting, and the concrete core to prevent any leakage at these places. The only difficulties in securing tight specimens occurred with the lean concrete mixes, in which pockets frequently occurred at the edge of the core, and with specimens submitted to dry curing con-

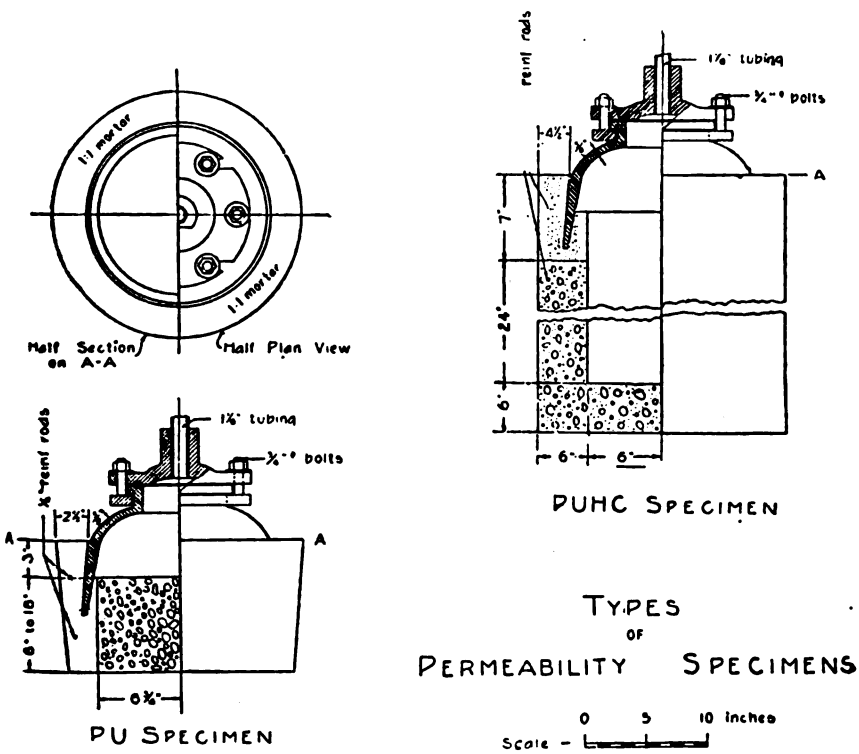

FIG. 1.

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ditions, in which cracks sometimes occurred between the core and the shell.

Tests on specimens made entirely of the 1 : 1 mortar used for the shells showed that such a mortar is practically impervious to water; the leakages were so slight that they would not affect any results determined from the concrete. Of two such specimens tested at the age of 21 days, one absorbed 0·0022 gallon in 304 hours and the other 0·0003 gallon in the same period. A third specimen, tested when four months old, showed no leakage in 304 hours. The bottoms of these specimens were covered with impervious building paper, so that the leakages were through or into the sides only.

Since the "PU" specimens could be used only for observing the permeability of concrete when the water flow paralleled the direction of placing the concrete, five batches were made into "PUHC" specimens in order to give information regarding the flow of water perpendicular to the direction of placing the concrete. This type of specimen (*Figs.* 1 and 2) was similar to the "PU" specimen, having the same general shape and the same kind of iron casting, but the side walls of the specimen to within 2 in. of the casting were made of concrete rather than mortar. Thus nearly all the water, under pressure, permeated through the side walls perpendicular to the direction of placing. In the "PUHC" type of specimen the inside area of concrete exposed to the water was considerably less than that exposed to air.

The apparatus for measuring the flow of water into the specimens is shown in *Fig.* 3. Measurement of the flow of water into the specimen was made by a water column or "permeability tube" attached to each specimen. Each water column consisted of an iron pipe 1⅛-in. in diameter, a ½-in. glass water gauge, and a scale (*s*) graduated to read the quantity of water in the tube. The smallest divisions of the water gauge represent intervals of 0·001 gallon; and, by the use of a sliding target (*t*), readings may be interpolated to 0·0001 gallon. The water gauge was connected to the pipe at the top and bottom.

The permeability tube was attached to the specimen by means of a heavy iron flange. A rubber ring was inserted in a groove between the flange and the bonnet to ensure a tight joint when the connecting bolts were turned down. A small cock (*p*) was attached to the tube just above the flange so that water might be forced into the column before commencing the test. The air tube (*c*) from the pressure tank entered the top of the permeability tube, where a valve (*v*) was provided for cutting off the supply when the water column was not in use. The tubes from the battery of water columns were joined to a manifold which was supplied with air by a pipe from the pressure tank (*r*). The tank was of relatively large capacity, when compared with the volume of the pipes and tubes, so that leaks in the specimens causing a lowering of the water levels did not materially decrease the air pressure.

No provision was made for catching and measuring the water escaping from the free surfaces of specimens, but whenever such leakage was visible it was recorded.

The sand for the specimens was air-dried before use unless otherwise specified. In the batches made with wet sand the air-dry sand was placed in the mixer and wetted with a given amount of water; the mixer was run until the sand was uniformly wet before the other materials were added. The surfaces of the particles of stone were usually dry at the time of making the concrete. Occasionally specimens of stone were selected from the storage pile and tested for moisture content so that the total per cent. of water in the mix could be estimated.

A converse rotary sieve, set at a slope of ½-in. per foot and run at a speed of 11 r.p.m., was used in grading the stone. This sieve separated the stone into the following sizes: 0 to ¼, ¼ to ½, ½ to ¾, and ¾ to 1¼ in. Graded coarse aggregate was used in all of the concretes made.

The moulds were filled as soon as possible after the concrete was mixed. A layer of 1 : 1 shell mortar was first tamped into the space between the cylinder and the outer mould. Equal portions from each wheelbarrow of concrete were then taken to make up a 3-in. layer inside the cylinder. Thus successive layers of mortar and concrete were placed and tamped or puddled until the required thickness was obtained. Dry mixes were tamped with an iron tamper having a face 3 in. in diameter. Medium and wet mixes were puddled with a

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½-in. iron rod, and wet mixes were poured into place without tamping or puddling. Two reinforcing rings were placed in the mortar shell to take up stresses developed in curing. Care was taken to lift the inner cylinder vertically to avoid distorting the concrete core, and the entire specimen was tamped and jarred in order to bring the core and the shell into good union. The top was trowelled off even with the rim of the outer mould, care being taken not to brush any of the shell mortar over the concrete core. To prevent drying of the surface, a cement sack soaked in water was placed over each specimen.

On the day after moulding, the tops of "PU" specimens and the outer surfaces of "PUHC" specimens were scrubbed with wire brushes to remove laitance and rich mortar. Usually the specimens were removed from the moulds on the second day after pouring. The sand was washed out of the bonnets and the inner surfaces of the specimens were chipped with a heavy steel bar provided with a chisel end. The cleaned specimens were usually filled with water, covered with sacks and sprinkled twice daily until tested. The outer mould of each "PUHC" specimen was stripped on the day following fabrication and the outside of the specimen was swathed in wet sacks. Four or five days after moulding, the core was removed and the interior surface of the specimen was chipped. The test pieces were then inverted, filled with water, and covered with sacks which were kept damp throughout the curing period.

In considering the results of these permeability tests it should be borne in mind that the tube readings represent seepage of water into the concrete and necessarily include both absorption and flow through it.

In specimens exhibiting considerable leakage at the free surface the absorption

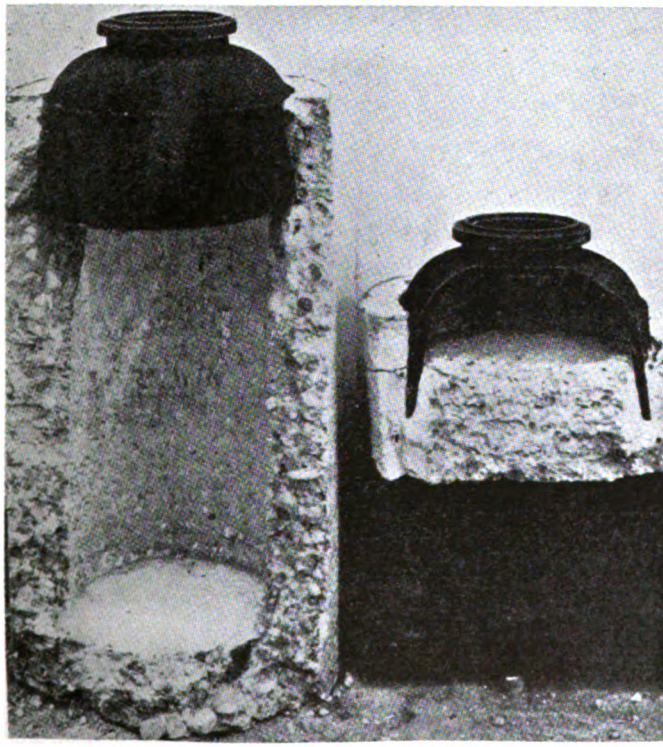

Fig. 2.—Longitudinal Sections of "PU" and "PUHC" Specimens.

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is relatively small, but in specimens appearing dry on the free surface its effect is of more importance. However, since all specimens were filled with water several hours before they were connected to the apparatus, the absorption during testing was small. Also from the time-leakage curves it appears that the rate of flow into the specimens was most rapid during the first ten or twenty hours of the test and became nearly constant during the 40–50 hour period. This decrease in rate of flow was undoubtedly due to the saturation of the pores and to plugging of the capillary ducts or tubes by silt and bacteria from the water. To determine the effect of bacteria in the water in retarding the rate of flow, two pairs of specimens which had exhibited approximately equal permeability were re-tested, one pair being filled with water from the regular supply, the other with a solution of formaldehyde. There was, however, no appreciable difference in leakage in the second test. Apparently the bacterial action was negligible. Tests made with this apparatus, while some experiments on gravel concrete were in progress, demonstrated that evaporation from the free surface of specimens exhibiting no visible signs of leakage would account for the continuous and rather uniform rate of flow which such specimens exhibit after being under pressure twenty or more hours. Furthermore, it was demonstrated that the rate of flow was more rapid for low humidities than for high humidities; and it was also shown that leakage at the free surface was more readily detected by the eye when the humidity was high than when it was low.

It is believed that this method of testing the permeability of concrete to water furnishes a safer index of water-tightness than methods in which leakage is measured by the amount of water escaping from the free surface or by visual examination of that surface. It also appears probable that the proportion of the flow into the concrete, which is absorbed by it, is relatively small after the first twenty hours of the test period. Therefore, it seems evident that, for specimens exhibiting little if any visible signs of leakage, the average rates of flow for the 20–50 and 40–50 hour periods are better indices of true leakage than the values of the 0–50 hour period.

One batch of 1 : 1½ : 3 and one batch of 1 : 2 : 4 concrete were tested to show the effect of age on permeability. None of the specimens showed any discoloration or dampness of the bottom during the 14-day or subsequent tests. The concrete of both batches was so dense that the rate of flow of water into the specimens was not high enough to cause visible dampening of the free surface.

A table and some curves show that the rate of flow, at any given age, was consistently less for the 1 : 1½ : 3 concrete than for the 1 : 2 : 4 mix. The computed rates of leakage for periods of 0–50, 20–50, and 40–50 hours generally are less for each successive test. However, the decrease is not large after 64 days for the 1 : 2 : 4 and after 29 days for the 1 : 1½ : 3 mix. Thus it appears that the imperviousness of a medium or rich mix is not increased materially with age after it has been given a proper curing period. A comparison of batches shows that the rate of flow over a given period is about the same for specimens that had been previously subjected to water pressure as for similar specimens of the same age that were tested for the first time.

Some of the specimens were subjected to freezing and weather conditions. In submitting the specimens to freezing conditions they were placed in the chamber of a refrigerator which was operated by means of an ammonia compressor. The test pieces were stored in the permeability room and kept full of water until the start of the freezing period, when they were emptied. The first run in the freezer extended over four days; the average temperature for the period was 3° F., the second and third days being at 0 degrees. The lowest temperature obtained was – 10° on the last day of the freezing process. After removal from the refrigerator the specimens were allowed to thaw out in the laboratory; they were submitted to water pressure and tested one day after removal from the freezer. A second and third run in the freezer followed, the treatment of the specimens being the same as before. The length of the second run was 6 days, the lowest temperature being 10° F. The average temperature recorded was 12° F. The third freezing period lasted only three days at an aver-

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age temperature of 5° F., the lowest temperature was 2° F.

The results do not indicate any notable change in the permeability of the specimens due to freezing. However, comparing the leakages obtained after freezing with values that might be expected at the same age for specimens not frozen by interpolating between the results of the 64 and 188 day tests, it is apparent that the effect of freezing was to increase slightly the rate of flow.

After the last freezing two test specimens of the 1 : 1½ : 3 batch were placed out of doors resting on their curved surfaces where they were subject to all weather conditions. They remained outside for three and one-half years, and were then thawed out and tested. The rates of leakage for these specimens were about the same as for the same set at 14 days, showing that the effect of the weathering agencies was to increase considerably the permeability.

Specimens 12 and 18 in. thick were made of 1 : 2 : 4 (by volume) and 1 : 9 (by weight) concrete to compare with similar 6-in. specimens. The data available is not complete enough to lead to definite conclusions, but it is apparent that increasing the thickness of the concrete did not materially affect the rate of flow into the specimens. Numerically the average leakage rates are somewhat greater for the 12 and the 18-in. specimens than for those made 6 in. thick. Probably this is due to the greater amount of pore space in the thicker specimens and to compression of the pores in the exposed surface of the 6-in. specimens due to bending under testing pressures.

A table is given summarising the results of tests on batches of concrete of medium consistency containing differ-

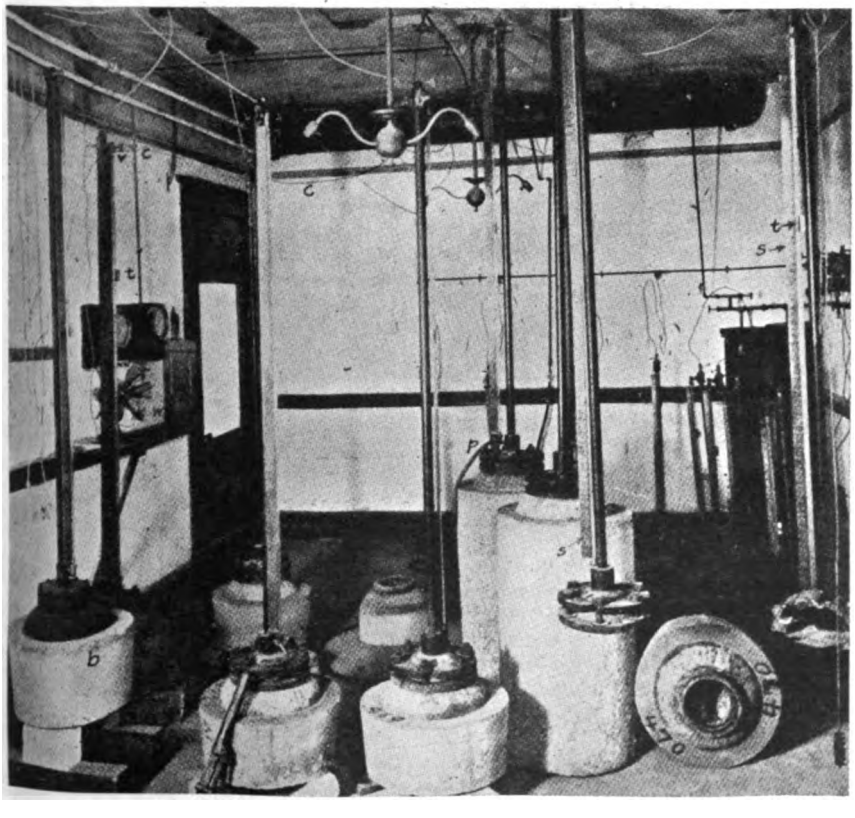

FIG. 3.—PERMEABILITY APPARATUS.

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ent proportions of cement. In these batches the cement varied from 10 to 18·5 per cent. of the total weight of dry materials. No visible indications of leakage were noted for any of the specimens from batches with 12·5 per cent. of cement or richer, but all of the speciments with 10 per cent. were moist in spots, discoloured, or actually dripping. An increase in the cement content above 15 or 16 per cent. of the total weight of dry materials did not effect a corresponding increase in water-tightness. The broken stone concretes exhibited more than twice the rate of flow of gravel concretes having like ratios of cement to aggregate.

The importance of so combining the aggregate in a concrete mix as to secure the highest density possible has been recognised for some time, and many tests have been made with graded sands and coarse materials to study the influence of density. In this series of tests the stone was screened into five sizes and recombined in proportions that would give a mechanical analysis curve approximating a straight line. The different sizes of stone were kept separate until parts were weighed out for each batch at the time of mixing; this was done to avoid the segregation which would occur in a mixed stock pile of different sizes.

Experience shows that mixes graded to conform to the "Fuller" curve are generally tight and dense. Mixes having curves with the lower portions below the "Fuller" curve are too harsh to be worked into a water-tight concrete. Although the number of tests is insufficient to establish definitely the best method of proportioning to secure a water-tight concrete, it appears from what has been done that lean mixes graded to Fuller's curve and sufficiently mobile to ensure against pocketing during placement can be made water-tight. Whether true economy lies in using a lean mix containing a carefully graded aggregate or in the use of an arbitrarily proportioned mix richer in cement will depend upon the quantity of concrete to be placed, the cost of cement, and the extra cost of securing aggregate graded to meet the requirements of Fuller's curve.

To determine the effect of length of wet and dry mixing period and of the use of wet sand, tests were made on batches of 1 : 1½ : 3, 1 : 2 : 4, and 1 : 9 proportions. The tabulated results show that for 1 : 1½ : 3 proportions and a mixing period of ¾ to 5 minutes the rate of flow of water into the concrete was not affected by total time of mixing. The batches made with wet sand, however, were somewhat more permeable than those made with dry sand.

For leaner mixes the length of the mixing period had a marked effect. It is seen from the data of the 1 : 2 : 4 batches that concrete mixed two minutes leaked less rapidly than the specimens of the three-quarter minute batch. Similarly there was a gain of water-tightness when the mixing period was increased to five minutes. The effect of wet sand in increasing permeability was also more notable in this set of specimens than in the 1 : 1½ : 3 batches.

The 1 : 9 batches proportioned 1 : 3·6 : 5·4 show, uniformly, that lengthening the mixing period decreased the permeability. The same effect is shown by the data from the 1 : 3·8 : 5·2 specimens.

The results from sixteen batches of concrete show the variation of the rates of flow of water into specimens of dry, medium, wet, and "soupy" consistency and of four different proportions. The dry concrete test pieces were tight enough to hold the pressure of water in testing, but those of 1 : 9 proportions leaked very rapidly. Difficulties were encountered in maintaining the pressure on the batches of "soupy" consistency, because the rate of flow for these specimens was much more rapid than for the drier mixes of the same richness. The 1 : 9 mixes of "soupy" consistency leaked so rapidly that constant pressure could not be maintained, and therefore they were not tested for a full period of 50 hours.

In all cases the specimens of medium consistency were more impervious than the others of the same richness.

Specimens of 1 : 1½ : 3, 1 : 2 : 4 and 1 : 9 concretes were submitted to various curing treatments. The specimens of 1 : 9 concrete were affected much more seriously by adverse curing conditions than the richer mixes. Removal from the moulds after two days followed by dry curing at room temperature increased the rate of flow 10 to 30 times over the rate obtained when the concrete was sprinkled twice daily for a month. With

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specimens left in the moulds for 7 or 14 days, followed by air curing, the leakages were correspondingly less than for the specimens removed from moulds after 2 days, but still considerably higher than for the concrete sprinkled a month. A comparison indicates that the permeability of 1 : 9 concrete which had been sprinkled for one month was not materially affected by subsequent drying in air at room temperature for an equal period. Leakage was, as expected, considerably reduced by doubling the sprinkling period.

The results for 1 : 2 : 4 and 1 : 1½ : 3 concrete lead to conclusions similar to those derived from the 1 : 9 mixes. Specimens that were sprinkled until tested were in all cases most nearly impervious. Drying in air at room temperature had no particular effect on the rate of flow after the concrete had been properly cured by sprinkling for 28 days, but early removal of the moulds and curing in air for an equal period, without sprinkling, notably increased the leakage. Late removal of the moulds, which prevented ready evaporation of moisture from the concrete, was effective in reducing the undesirable effect of air-curing. Heating of 1 : 2 : 4 and 1 : 1½ : 3 concretes, after 23 or 51 days of sprinkling, reduced the rate of flow markedly.

In general, the rate of flow into the 1 : 2 : 4 or 1 : 1½ : 3 concrete increased with the per cent. of weight lost in curing. None of the 1 : 1½ : 3 mixes under any of the conditions tested gave evidence of leakage on the free surface.

Except in a few irregular cases the rate of leakage into stone concrete specimens of this series decreased with the continuation of the test. The greatest reduction in rate of flow was noted for specimens which were dried out before the start of the test. It is logical to assume that these specimens absorbed water rapidly from the permeability tubes during the first hours, so that the 0–50 hour leakage would be considerably higher than the rate for 20 to 50 or 40 to 50 hour periods. In some cases high rates of flow were obtained without any visible leakage at the free surface.

CONCLUSIONS.

From the permeability tests on gravel and stone concretes which have been made at the University, the following conclusions are drawn :

(1) None of the concretes tested was absolutely water-tight, if continuous flow into the specimen is considered proof of permeability. The specimens of rich mixes, when submitted to proper curing conditions, were so impervious that no leakage was apparent at the free surface. Such concretes can be considered water-tight for ordinary purposes.

(2) The cement content of a given mix appears to be the most important factor in determining the degree of impermeability. Other factors remaining constant, the water-tightness of concrete made with well-graded aggregate increases with the proportion of cement in the mix. In the mixes tested, this increase in water-tightness was very marked as the cement content was raised until the ratio of cement to aggregate, by weight, was 0·15. Further increase in cement content was attended by a relatively small increase in water-tightness. For the broken stone concretes of these tests a ratio of cement to aggregate, by weight, of 0·17 or more gave a water-tight mix, whereas a cement-aggregate ratio of 0·14 was sufficient for the gravel concrete mixes. Mixes containing high proportions of cement were much less affected by variations in consistency, time of mixing, and grading of aggregate than lean mixes.

(3) The ratio of the volume of cement to the volume of air plus water voids, or cement-voids ratio, is a valuable measure of both strength and water-tightness of concrete. The strength increases nearly in direct proportion to the cement-voids ratio for the mixes tested. Water-tightness in the mixes of broken stone concrete was obtained when the cement-voids ratio was 0·55, or above. With the gravel-concrete mixes a cement-voids ratio of 0·45 gave similar results.

(4) The water-tightness of mixes containing less than 14 per cent. of cement to total dry materials, by weight, is much affected by variations in consistency, curing conditions, placement, and grading of the aggregate. It is also affected, but to lesser extent, by variations in the time of mixing, and by the moisture content of the fine aggregate. Lean concretes made of angular broken stone are more

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sensitive to the effects of these variables than similar mixes containing round pebbles.

(5) Maximum water-tightness obtains for a limited range of the water-cement ratio $\frac{w}{c}$. For the rich mixes this range is much wider than for lean mixes.

(6) A plastic consistency which gives a slump of 1 to 3 in. when tested in a 6 by 12-in. cylinder appears to be best suited to produce an impervious concrete with a minimum of cement. In order to secure maximum density it is better to use somewhat more water than above indicated rather than a lesser quantity; but the use of excessive amounts of water, which is usually accompanied by segregation of the aggregate, greatly increases the permeability of lean mixes.

(7) When conditions of placement require the use of a more fluid mix than recommended above, increased fluidity may be obtained, without sacrifice of imperviousness, by raising both the cement and water contents in the same proportion.

(8) To secure a high degree of imperviousness, concrete should be cured in a moist atmosphere or under water. At temperatures of 60° to 75° F., the moist-curing period should be maintained for two weeks to a month after moulding. Lean mixes or thin sections require longer wet-curing periods than rich mixes or thick sections. When it is impracticable to keep the work moist after removal of the moulds, and the humidity is low, provision should be made to keep the moulds in place in order to avoid evaporation. The effect of dry curing appears to be much more detrimental to water-tightness than to compressive strength.

(9) Thorough mixing is of great importance in securing the homogeneity necessary in water-tight concrete. Considering only the consistencies more commonly used, slumps 1 to 10 in., it appears that lean mixes made with damp sand require a somewhat longer mixing period than when made with dry sand. With a mixer of the type used, a mixing period of one and one-half to two minutes after all materials have been placed in the machine is required to secure thorough mixing of lean concretes. Materials for such concrete should be mixed dry from 15 to 30 seconds before water is added.

(10) With lean mixes very much greater care must be exercised during placement to secure sufficient homogeneity so that they will be impervious when the direction of flow is parallel to the bed. The rich mixes of 1 : 1½ : 3 proportions exhibited practically the same degree of water-tightness for flow normal to or parallel to the bed.

(11) The results show that lean mixes in which the materials have been proportioned in accordance with Fuller's mechanical analysis curve can be made very impervious provided it is possible to use a plastic consistency and to secure proper methods of placement and curing. Therefore, when a construction involving a large yardage of concrete is being planned, the properties and the costs of lean mixes made of specially graded aggregates should receive due consideration.

(12) For making water-tight concrete, well graded gravel of good quality is superior to broken stone of like quality and grading.

(13) For properly placed and cured broken-stone or gravel concrete the permeability decreased as the compressive strength increased until the latter reached 2,500 lb. per sq. in. Concretes of this strength were water-tight.

(14) For the concretes tested, increasing the thickness above 6 in. did not materially affect the leakage at pressures of 40 lb. per sq. in. With the more variable conditions of mixing, placing, and curing encountered in the field, sections thicker than 6 in. would undoubtedly be required for water-tightness.

(15) The permeability of rich concrete cured in moist air for one month and subsequently protected from the weather decreases with age, but the decrease is negligible after the concrete becomes one month old.

(16) When concrete is subjected to freezing its permeability appears to be increased slightly. Weathering agencies, such as combined freezing and thawing, wetting and drying, and variations in temperature seem to produce a marked increase in permeability. The effect of these agencies must be more thoroughly investigated, however, before their magnitude can be stated.

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THE FOREMAN'S GUIDE TO CONCRETE.—IV.

A MANUAL FOR ALL ENGAGED IN CARRYING OUT CONCRETE AND REINFORCED CONCRETE STRUCTURES.

By ALBERT LAKEMAN, M.S.A., M.I.Struct.E.

NEUTRAL AXIS.

A KNOWLEDGE of this expression is very important to the person in charge of reinforced concrete work. In a simple beam which is supported at each end and subject to a load, the top portion of the beam will be in compression and the bottom portion will be in tension. This can be easily proved by taking a small piece of wood and putting it on two supports as shown in *Fig.* 22 and placing a weight on the centre sufficient to cause it to break, as indicated. Upon examination of the broken wooden beam it will be seen that the bottom fibres have been torn apart by stretching or tension and the top fibres are crushed by compression.

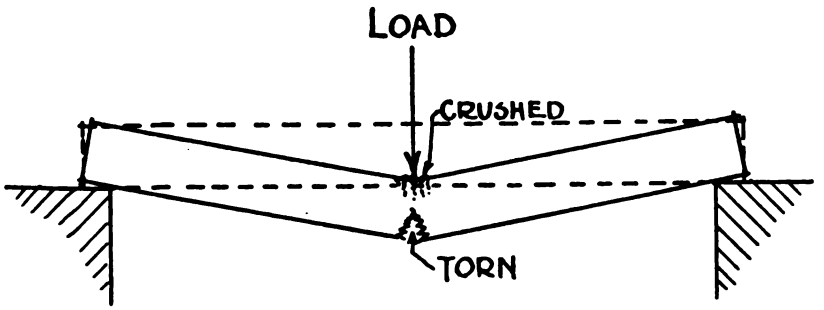

FIG. 22

Now, it will be obvious that if the top portion of the beam is being crushed or shortened by compression and the bottom portion is being torn or stretched by tension there must be some line or point at which the stress changes, and at this line there cannot be any stress as there is no tendency either to shorten or stretch. This line, not being subject to compression or tension, is neutral, and is known as the "neutral axis," while the layer of the beam in which it is situated is known as the "neutral layer." For the moment no consideration is being paid to the question of shearing stress, as this will be dealt with separately and it is necessary to avoid possible complications.

The position of the neutral axis is quite dependent on the change from the compression to tension stress, and the expression "neutral" means neutral as far as these are concerned. The change in the stress from one kind to another is gradual throughout the depth of the beam; the greatest compression will be in the top fibres, and from that point downward the stress gets less and less until the neutral layer is reached, where it disappears entirely. The greatest tension is in the bottom fibres, and this gradually diminishes upward until the neutral layer is reached, where again this stress becomes nil.

It was previously stated that the amount of the compression stress in a beam subject to a load and the amount of the tension stress are equal, and in the case of a beam which is rectangular

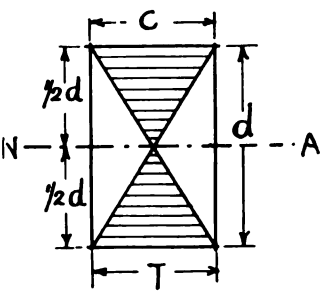

FIG. 23

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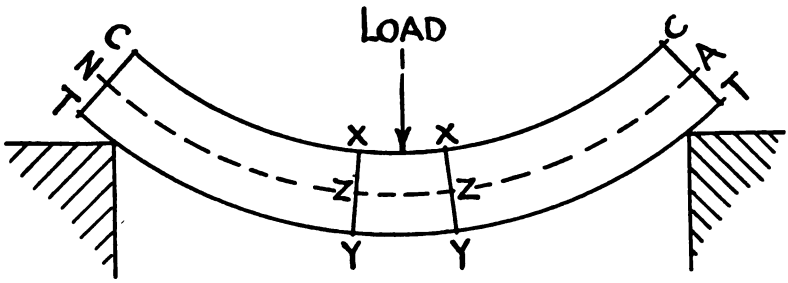

FIG. 24

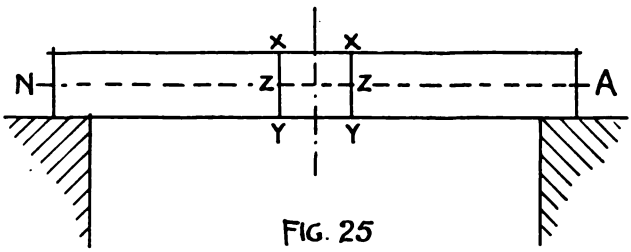

FIG. 25

in section and composed of a material having an equal resistance per square inch to compression and tension the neutral layer will occur at the centre of the depth. This is shown in *Fig.* 23, where the neutral axis is indicated by the line *NA* situated at one-half the depth from the top and bottom. The variation in the stress from the top and bottom, where it is greatest, to the neutral axis, when it is nil, is shown by the shaded portions, and the length of the horizontal line at any point will give the relative stress at that point. The importance of a knowledge of the position of the neutral layer will be appreciated when it is considered that it may be necessary to cut a hole through the beam and that by making the hole where the neutral axis occurs the beam will not be weakened to any appreciable extent, whereas if the cutting away is done at the top or bottom of the beam, (shown in *Fig.* 23) the section will be seriously reduced where the greatest resistance is necessary.

The conditions in a reinforced concrete beam are different from those in the type here dealt with, as it is composed of two materials having different resistance values; this will be dealt with when the term has been fully explained. The reason for the shape of the old tusk-tenon joint used for trimming one wooden beam into another will be appreciated, as it will be seen that the material cut away in the supporting beam is arranged with reference to the variation of the stress and the position of the neutral layer.

The variation in the stress can be shown in another way by considering a loaded beam which is bent or deflected considerably, as shown in *Fig.* 24. The bending of the beam is very much exaggerated in order to illustrate the tendency for the top part to be shortened by the compression and the bottom portion to be stretched by the tension. As this beam when bent is part of a circle it will be obvious that the length of *C–C* is less than the length of *T–T*, and if measurements were taken before and after the load was applied it would be found that the length *N–A* was unaltered, whereas *C–C* had become shorter and *T–T* longer. The length of *N–A* is not affected by the load because this is the position of the neutral axis, or neutral layer, and no stress is caused at this point. If a piece of the beam, say, at the centre, is marked with two points at the top and two at the bottom, as shown by *X–X* and *Y–Y* in *Fig.* 25, the distance between these two pairs of points being equal so as to give two parallel lines on the face of

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the beam as drawn, then after bending it will be seen that these two lines are no longer parallel, as they will both radiate from the point which represents the centre of the circle of which the beam is a portion. The distance *Z–Z*, however, which represents where these lines cross the neutral axis, will remain unchanged, and we can set this part of the beam out to a large scale as shown in

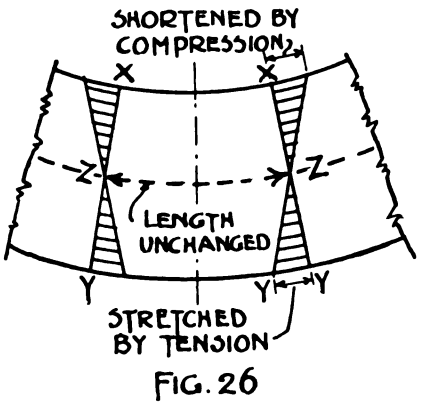

Fig. 26. Two parallel lines can then be drawn through the points *Z–Z*, as indicated, and the variation in the shortening and stretching, which is also the variation in the stress, can be seen by the shaded portions, which show the gradual diminishing from the top and bottom to the neutral axis, as explained in connection with *Fig.* 23.

The neutral axis, or neutral layer, is therefore the position at which neither compression nor tension exists, and in a plain ordinary rolled steel joist this will be situated at the centre of the depth. In the case of a reinforced concrete beam there is a neutral axis, but the position of this will not generally be at the centre of the depth because the two materials used have different strengths and different elasticity values. It has previously been shown that the tendency to shorten the top portion of the beam and stretch the bottom portion is nil at the neutral axis, and it gradually increases as the distance from the neutral axis increases; and as the resistance of the beam must be equal to the stress it will be economical if the resisting material can be concentrated at the point where the stress is greatest in order to enable it to develop its full strength, as the material close to the neutral axis is not called upon to do the maximum amount of work.

If one looks at the section of a rolled steel joist as used in building work, it will be noticed how the greater part of the steel is provided in the flanges and kept as far from the neutral axis as possible. In a single-reinforced concrete beam the steel is kept in the lower surface as far from the neutral axis as possible, as it would otherwise be wasted.

Now, it has been stated that the compression stress and the tension stress in a beam under load will be equal to each other, and if this is so then the resistance to compression and tension must also be equal. The maximum safe resistance of the concrete, however, is limited to 600 lbs. per sq. in., and if the steel is at the same distance from the neutral axis as the concrete the maximum resistance that could be developed would be 600 × 15 (to allow for the difference in elasticity as previously explained), which only gives a value of 9,000 lbs. The steel can, however, be safely stressed to 16,000 lbs. per sq. in., and consequently the neutral axis occurs nearer the top of the beam, so that the stress on the steel is increased without overstressing the concrete.

The exact position of the neutral axis in a reinforced concrete beam is calculated by the designer, as it depends on the depth of the beam, the safe stress on the concrete, the safe stress on the steel, the proportion of steel to concrete, and the ratio of the elasticity of the two materials. It is not proposed to show the method of calculation as this is beyond the scope of these present notes, but in the case of a single-reinforced concrete beam where the economical amount of steel is provided the position of the neutral axis will be 0·36 *d* from the top surface of the beam, or roughly one-third *d* where *d* equals the depth of the beam in inches.

As an example, if a beam is 18 ins. deep the position of the neutral axis will be 0·36 × 18 = 6·48 in. or nearly 6½ ins. from the top surface. In rough calculations for preliminary designing, or for getting the approximate position of the neutral axis in a beam after it has been constructed, a very common practice is to take one-third of the depth of the beam from the top surface, and in the example just mentioned one-third of the depth equals 6 ins., which is very near

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the figure given above of 6½ ins. Also, when making accurate calculations the depth in inches is taken as the theoretical depth measured from the top surface of the concrete to the centre of the reinforcement, whereas in the rough calculation after a beam has been constructed the depth in inches is generally taken as the total depth from the top surface of the concrete to the underside of the beam; and one-third the total depth will be very close to the value of 0·36 *d* where *d* is the theoretical depth.

The section of a simple reinforced con-

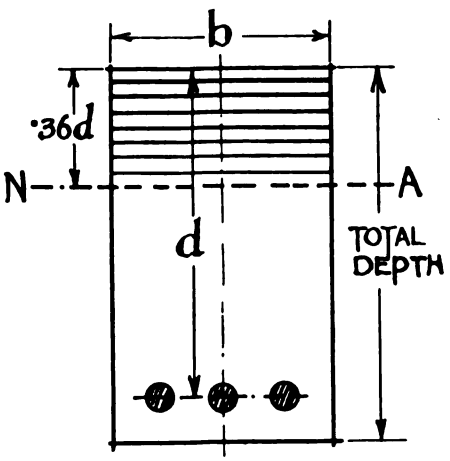

FIG. 27

crete beam is given in *Fig.* 27, and the position of the neutral axis and the difference between the depth as used in calculations and the total depth are indicated. It will be noticed that the concrete above the neutral axis is shaded, and this is the only portion which is called upon to resist any stress. The value of the concrete in tension is neglected, and as the whole of the beam below the neutral axis will be subject to tension stress no allowance is made for the value of the concrete below the axis and the steel has to provide all the resistance.

This is a very important point, because it may be necessary to cut holes through the beam for the purpose of bolting up hangers or for passing a small pipe through the concrete. It will clearly not be good practice to cut these holes at any point above the neutral axis, because the resistance of the concrete to compression will be reduced. If the holes are cut below the axis, however, the material which is cut away will not affect the resistance of the beam to tension, because the concrete below the neutral axis is not designed to take any of the tension and is not theoretically required as far as resistance to the bending moment is concerned.

It will be seen that the conditions in the case of a reinforced concrete beam are quite different from those where the beam is composed of one material only, both as regards the position of the neutral axis and the effect of cutting holes in the beam after it is in position.

To summarise the important points in connection with the neutral axis in a single-reinforced concrete beam it may be stated that—

(1) The neutral axis will be situated at about one-third the depth of the beam from the top surface, and
(2) any holes required through the beam should be provided *below* the neutral axis.

The question of beams with double reinforcement, continuous or fixed beams, and tee beams will be considered later on, together with some notes on the variation of the shearing stress.

REACTION.

The explanation of the term "reaction" will not possibly be of great importance to the foreman in charge of reinforced concrete work, but as the expression is one which is used repeatedly by the designer, a note on the meaning will not be out of place here.

A load or force in constructional work is always tending to cause an action, namely, to break the part of the structure that is called upon to offer resistance, and generally speaking the action, if not counteracted, would result in a falling downward of a part or the whole of the building. In the case of a load on a beam the strength of the beam must be sufficient to prevent it breaking, but apart from this the beam must be kept in position by the supports at each end; thus the resistance of the beam itself is not the only point that has to be considered. In other words, the action of the load is to push the beam downwards.

Now, this tendency of the load is

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resisted by the supports at the ends of the beam, which react, and by their resistance keep the beam up in its position. The amount of resistance that has to be given by a support is known as the "reaction," and it must always be equal to the amount of the load which is brought on to the support by the beam, or failure will occur. The action of the load is downward, and the reaction at the supports is a resistance which is equivalent to an upward force, although it is not actually a force but is the strength of the material at the support that is called upon to exert itself under the action of the load.

Action and reaction are directly opposed to each other, and the reaction is non-existent until called into being by the action. A diagram is given in *Fig.* 28 which will illustrate the relation between the load and the reaction. A beam is shown which carries a concentrated central load of 10 tons, and the ends of this beam are supported on reinforced concrete columns. The load is up in the air and it must be transmitted to the ground through the medium of the beam and the columns. In order to do this the load theoretically travels along the beam from the point of application in each direction, as shown by the arrows, and as the load is central and the distance to both supports is equal the amount that travels to each support will be equal, and in this case will be 5 tons.

As the amount of the load that reaches the support at *A* is 5 tons the amount of resistance given by the support, or the *reaction* as it is called, must also be 5 tons. The resistance of the column may possibly be greater than 5 tons, but the reaction is always equal to the amount of the load that comes on the support, and when making calculations the designer must first ascertain the value of the reaction and then provide for the necessary material to give this reaction.

When the support is sufficient to carry the load the latter travels downward through the support until it reaches the soil, as at *B* in *Fig.* 28, and here the reaction is provided by the bearing capacity of the ground.

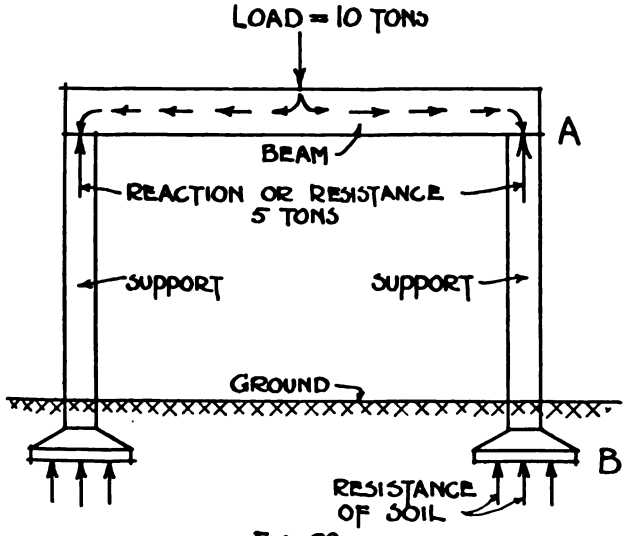

FIG. 28

In the case of central concentrated loads and uniformly distributed loads the reactions at the supports are equal to each other, and as the total value of the two reactions must be equal to the total load the value of each reaction will be one-half of the load. As the load shown in *Fig.* 28 is 10 tons, then each reaction will equal 5 tons, and the action of the load is therefore balanced or counteracted by the reactions.

When the load on a beam is not central or uniformly distributed the two reactions will not be equal to each other,

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as the support which is nearer to the load will take the greater part of the load. For example, if a load of 10 tons is concentrated at a point which is one-quarter of the span from one support and three-quarters of the span from the other support, then the support which is near the load will have to carry three-quarters of the load of 10 tons, which equals 7½ tons, and the other support will only take 2½ tons. The reactions therefore will be 7½ tons and 2½ tons. This difference is explained by the fact that the load has to travel three times as far along the beam to reach one support as it has to the other.

Should the reader be familiar with the three orders of levers he will recognise that this is an example of the third order, and the reactions can be calculated for any condition of loading in a very simple manner.

There is one point which will appeal to the foreman in connection with this item, and that is the necessity of getting a good even bearing for a beam or slab on the support and a good connection between a beam and the supporting columns, as the whole of the load has to pass through the support or connection and a weakness at the junction of beam and wall or beam and column will possibly result in failure owing to this load not being properly transmitted to the support. This point is frequently considered in a casual manner only because the importance of the reactions is not fully appreciated.

(*To be continued.*)

Recent Patent Applications.

209,027.—Fortis Reinforced Concrete Safe Co., Ltd.: Reinforced concrete safes and strong-rooms.

212,575.—Fortis Reinforced Concrete Safe Co., Ltd.: Doors for strong-rooms and safes.

215,298.—C. W. Watson: Method of and machine for coiling wire reinforcements for concrete pipes.

217,227.—C. W. Watson: Machines for centrifugally moulding concrete structures, including pipes.

221,286: J. M. Ambrose: Shuttering support for concrete floors.

221,292.—Indented Bar and Concrete Engineering Co., Ltd. and R. L. Bendall: Construction of concrete roads, tracks, paths, buildings, tanks and bridges.

221,742.—V. Wikkula: Porous concrete.

222,609.—E. A. Smith: Concrete walls for houses.

222,746.—G. W. R. Nobel: Concrete blocks and slabs.

222,774.—C. Carnes: Pre-cast concrete blocks.

222,923.—R. Caccia: Lime-sand concrete and bricks and pipes.

223,059.—J. Bell & Co. Proprietary Ltd., W. R. D. Innes and M. S. Stanley: Reinforced concrete floors.

223,079.—H. G. O. Forain: Injection device for concrete plastering.

223,109.—J. F. Goddard: Expansion joints for concrete structures.

New Company Registered.

PHŒNIX BLOCKS, LTD. (20104). Registered October 18, 1924. Manufacturers of concrete slabs and blocks. Nominal capital £2,000. Directors: W. H. J. Eldridge, Courtfield Gardens Hotel, Courtfield Gardens, Kensington, S.W. (Managing Director), and J. D. O'Brien, junior.

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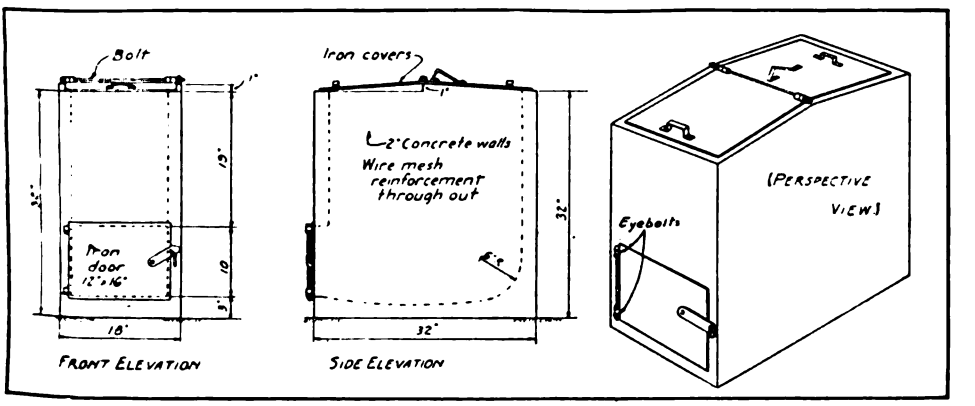
CONCRETE PRODUCTS.

A CONCRETE BUNGALOW.

[The Bungalow illustrated above has been built of breeze concrete block cavity walling, with breeze blocks 3 ins. thick for the inner and outer leaves and a 2-in. cavity. The bungalow is at Sandbanks, near Poole, and the blocks were made on a "Manelite" machine supplied by the Manelite Patent Concrete Machinery Co., Ltd., of Bournemouth.]



A CONCRETE DUST BIN.

[The advantages of a permanent material for the construction of dustbins, from the points of view of both hygiene and maintenance, are obvious. This design is extensively used in America, but can, of course, be varied to suit requirements. The drawing is from *Cement and Engineering News*, and needs no explanation.]

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Best British Portland Cement	per ton 58s. to	63	0
"Ferrocrete" Portland Cement	delivered London 10s. per ton	extra	
"Super-Cement"	d/d 4 miles Charing Cross, per ton 88s. to	93	0

BOARDING FOR SHUTTERING—		Sawn. s.	Sawn. d.	Wrot. s.	Wrot. d.
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in.	from £24 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £23 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅜ in. to 2⅞ in. Rounds	per cwt.	13	0
9/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds	,,	14	6
¼ in. Rounds	,,	15	6

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED WITH CLEAN WATER IN PROPORTION 1 : 2 : 4—

			s.	d.
Do.	do.	in foundation	per yard cube	48	6
Do.	do.	in columns	,, ,,	53	6
Do.	do.	in beams	,, ,,	53	6
Do.	do.	in floor slabs 4 in. thick	per yard super	5	8
Do.	do.	in floor slabs 6 in. thick	,, ,,	8	5
Do.	do.	in floor slabs 9 in. thick	,, ,,	12	3
Do.	do.	in walls 6 in. thick	,, ,,	8	8

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—

		s.	d.
From ¼ in. to ⅜ in.	per cwt.	25	0
,, 9/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2⅞ in.	,,	23	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1¼*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2¼*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.*

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.*

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	,, ,,	1	1½
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1½
Do. do. as last, in narrow widths	,, ,,	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Labourers on building works, 1/4½; Operators on concrete mixers and hoists, 1/5½.

[* This Data is specially compiled for *Concrete and Constructional Engineering*, and is strictly copyright.]

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CONCRETE "WHEELERS."

RAKE LANE, Clifton, near Manchester, is the only means of access to the industrial centre of Clifton Junction, and the paving throughout consisted of paved grit setts, the width between kerbs being 18 ft. The main difficulty in dealing with this road was that of being unable to divert the traffic, and Mr. E. L. Leeming, A.M.Inst.C.E., the engineer, decided to lay a length of about 300 yards along the centre portion of the road with concrete "wheelers" 18 in. wide and 7 in. deep, using B.R.C. fabric reinforcement 7 ft. wide, upon which between the "wheelers" the setts were to be paved and afterwards grouted in cement.

The method adopted consisted of constructing one wheeler with the reinforcement laid in the lower portion and the remaining width turned up to act as a kind of fence pending hardening of the single track, so that the traffic could run thereon when the remaining work began. The concrete was two-course work monolithic, the upper course being granolithic for wearing purposes. The work was done in sections so that traffic could pass at certain points. The second "wheeler" was constructed while the traffic ran along the other "wheeler," and within a few days the grit setts were paved hard on to the steel fabric and level with the concrete "wheelers"; the setts were afterwards grouted with chippings and cement. The outer edges of the wheelers were made good by paving the original setts for the most part, but in one portion a row of granite setts was laid.

Traffic was admitted on to the "wheelers" at the end of about two weeks. The traffic is somewhat intermittent, although in the course of a day the volume is quite considerable, and the concrete "wheelers" are now taking practically the whole of the wear of the road. The weak points of the work are the joints in the "wheelers" which were made at the end of each day's progress, but with special care they need cause no anxiety. On one section it appears that frost or continuous cold weather affected the hardening of the concrete, and some wear of the upper layer of about ½-in. thickness has taken place. It has not been necessary, however, after twenty months' wear to repair any length more than to place a little tarmac at the joints on the "wheelers" and here and there at the outer edge where it has been done in lieu of lifting the setts. The work was carried out at the worst season of the year, but with the exception of the portion which was affected by frost the "wheelers" are standing the wear as well to-day as when the road was opened. The width between the "wheelers" is 4 ft. and overall width between the outer edges 7 ft. No steel exists in the upper surface of the concrete. The road has had a severe test, as heavy consignments of lead, up to 13 tons on one lorry and trailer, have passed over the roadway. Moreover, the wheels of the whole of the traffic are of course concentrated on 3 ft. of the total width of the road.

CONCRETE "WHEELERS" ON AN INDUSTRIAL ROAD.

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A NEW BAR CUTTING AND BENDING MACHINE.

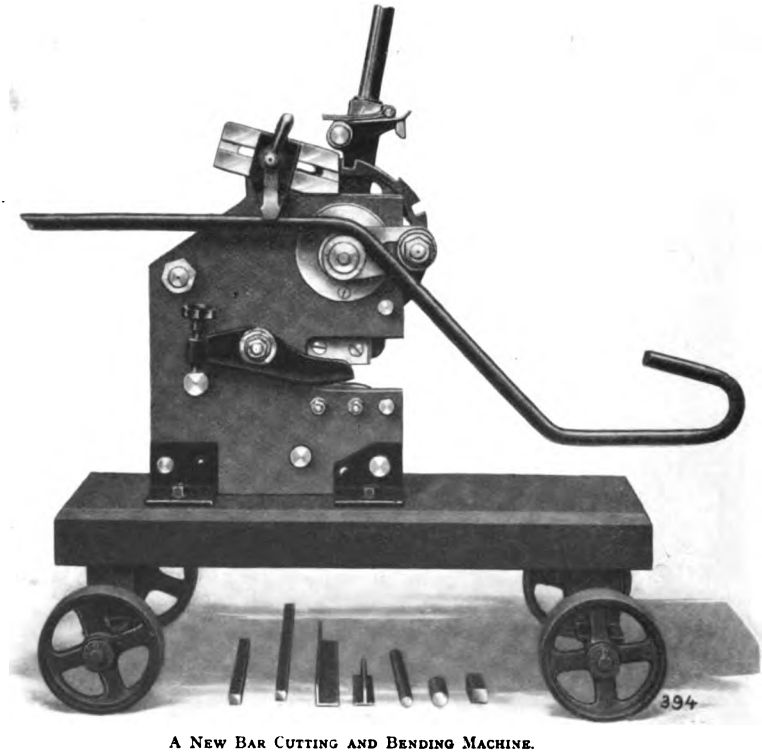
A NEW type of portable hand-lever shears with bending attachment for bars for reinforced concrete work has recently been put on the market by Mr. Henry Pels, of 40 Great Marlborough Street, London, W.1, and is illustrated below.

Although primarily designed for cutting reinforcement, the machine also crops squares, flats, angles and tees. By means of a bending attachment, forming part of the machine and worked off the eccentric shaft, bars can be bent into the required shape, as shown in the illustration.

Long bars are placed in position from the front of the machine, obviating the necessity of feeding long bars sideways through the shear blades. The "downholder" can then be brought into position, when the shears are ready for work. If profiled blades for rounds and squares are used, clean cuts with a minimum of distortion are obtained, but it necessitates feeding the bars in sideways, instead of placing them in position from the front of the machine. When bundling rounds, several bars can be sheared simultaneously at one stroke, providing that the total area of the small bars does not exceed the capacity of the machine.

When using the shears for bending work the bar is laid between the adjustable bending "downholder" and the raised bending roll. Using the hand lever, the bending roll moves in a circular way downwards, thus bending the rounds into the required shape.

The machine is made in four sizes. No. 1 will crop rounds and squares up to $\frac{3}{4}$ in., flats up to 2 in. by $\frac{3}{8}$ in., and angles and tees up to $1\frac{1}{2}$ in. by $1\frac{1}{2}$ in. by $\frac{1}{4}$ in.; this size machine will bend rounds of diameters up to $\frac{3}{4}$ in. The largest size of machine will crop rounds and squares up to $1\frac{1}{2}$ in., flats up to $3\frac{1}{8}$ in. by $\frac{3}{4}$ in. or 4 in. by $\frac{5}{8}$ in., angles up to 3 in. by 3 in. by $\frac{3}{8}$ in., and tees up to $2\frac{3}{4}$ in. by $2\frac{3}{4}$ in. by $\frac{5}{16}$ in.; and this machine will bend rounds up to $1\frac{1}{2}$ in. diameter. The two intermediate size machines will handle bars between these sizes.



A NEW BAR CUTTING AND BENDING MACHINE.

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PROSPECTIVE NEW CONCRETE WORK.

BARNARD CASTLE.—*Bridge.*—The Barnard Castle and Westmorland District Councils propose erecting a bridge at Cauldron Snout.

BEESTON.—*Houses.*—The U.D.C. has applied to the M.H. for sanction to erect 100 houses.

BEVERLEY.—*Houses.*—The T.C. has applied to the M.H. for sanction to erect 78 houses.

BIRMINGHAM.—*Reservoir.*—The Birmingham Water Committee proposes to erect a reservoir at Bartley.

BOURNEMOUTH.—*Houses.*—The T.C. has applied to the M.H. for a grant of £10,000 for the construction of 42 houses.

BOURTON.—*Road.*—The Gloucestershire C.C. proposes to lay a road at Bourton-on-the-Water.

BRAYFORD.—*Bridge.*—The Barnstaple R.D.C. has approached the C.C. with a view to erecting a bridge at Brayford.

BRIDLINGTON.—*Road.*—The T.C. proposes to construct a road from Fort Terrace to York Road.

BUDOCK.—*Sewage Disposal Works.*—The East Kerrier R.D.C. has applied to the M.H. for sanction to borrow £6,650 for sewage disposal works.

CARDIFF.—*Houses.*—The Corporation Housing Committee has applied for sanction to borrow £220,000 for building purposes.

CLECKEATON.—*Houses.*—The Spenborough U.D.C. has applied for sanction to erect 220 houses.

CRAMLINGTON.—*Bridge.*—The U.D.C. proposes to erect a bridge at Sandy Letch, at an estimated cost of £1,200.

CRICCIETH.—*Sewage Works.*—The U.D.C. has applied to the M.H. for sanction to borrow £4,758 for sewage disposal works.

CROOKHALL.—*Sewage Works.*—The Leadgate R.D.C. has applied to the M.H. for sanction to borrow £13,500 for sewage disposal works.

DARLINGTON.—*Houses.*—The R.D.C. has applied to the M.H. for a grant towards the construction of 50 houses.

DOLGARROG.—*Concrete Houses.*—The Colwyn Bay R.D.C. proposes to erect 100 concrete houses at Dolgarrog.

DUNDEE.—*Concrete Houses.*—The T.C. proposes to erect 500 concrete houses, at an estimated cost of £470 each.

FOLESHILL.—*Concrete Houses.*—The R.D.C. proposes to construct 500 concrete cottages.

GOBOWEN.—*Road.*—The Shropshire C.C. has a scheme in hand for the construction of a road at Gobowen, at an estimated cost of £9,150.

GOOLE.—*Water Tower.*—The U.D.C. has applied to the M.H. for sanction to borrow money for the construction of a reinforced concrete water tower.

GREAT YARMOUTH.—*Concrete Road.*—The Corporation proposes to lay a concrete road on the South Denes, at an estimated cost of £9,150.

GREENOCK.—*Houses.*—The Corporation Housing Committee is considering the question of building 250 houses.

HEBBURN.—*Houses.*—The U.D.C. proposes to build 100 houses.

HIGH PITTINGTON.—*Sewage Works.*—The Durham R.D.C. has approached the M.H. for a loan of £5,000 for sewage disposal works.

LEADGATE.—*Sewage Works.*—The U.D.C. has applied to the M.H. for sanction to borrow £13,500 for sewage disposal works.

LEEDS.—*Houses.*—The T.C. proposes to erect 500 houses at Meanwood, Crossgates, and Middleton.

LEYTON.—*Houses.*—The U.D.C. is considering erecting 130 houses.

MEXBOROUGH.—*Houses.*—The U.D.C. has applied to the M.H. for sanction to erect 100 houses.

NELSON.—*Reservoir.*—The Corporation Water Committee proposes to construct a reservoir at Ridgaling.

NEWCASTLE-ON-TYNE.—*Concrete Houses.*—The Corporation Housing Committee has decided to erect 300 concrete houses on the "Winget" system.

POULTON.—*Sewage Works.*—The U.D.C. has applied for sanction to borrow £14,350 for sewage disposal works.

REDCAR.—*Houses.*—The T.C. has approached the M.H. for permission to erect 100 houses.

ROTHERHAM.—*Concrete Bridge.*—The T.C. proposes to construct a concrete bridge in place of the old Chantry Bridge, at an estimated cost of £90,000.

RUGBY.—*Houses.*—The T.C. is considering a scheme for building 100 houses on the Eastlands estate.

SCARBOROUGH.—*Reservoir.*—The Cor-

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poration proposes to construct a reservoir at Prospect Mount.

SOUTHPORT.—*Houses.*—The Corporation proposes to erect 100 houses.

SOUTHWOLD.—*Groyne, etc.*—The T.C. is considering the question of erecting a concrete block revetment 200 ft. in length, together with a concrete groyne.

SPENBOROUGH.—*Houses.*—The U.D.C. has applied to the M.H. for sanction to erect 220 houses.

STANLEY (*Durham*). — *Houses.* — The U.D.C. has applied to the M.H. for permission to build 300 houses.

STANLEY (*Durham*). — *Road.* — The U.D.C. is considering a scheme for the construction of a road from Stanley to Newcastle.

SWAFFHAM.—*Houses.*—The U.D.C. is considering the question of erecting 50 houses.

TETBURY.—*Reservoir.*—The U.D.C. is negotiating for the purchase of land for the erection of a 1,000,000-gallon reservoir.

TRURO.—*Houses.*—The R.D.C. proposes to build 60 houses.

UFFCOLME. — *Bridge.* — The Tiverton R.D.C. proposes to erect a bridge over the ford at Rull, Uffcolme.

WARMSWORTH.—*Reservoir.*—The Doncaster Corporation is considering the question of erecting a reservoir at Warmsworth.

WEST KILBRIDGE. — *Reservoir.* — The Ayrshire C.C. has applied for a grant towards the construction of a reservoir estimated to cost £11,500.

WOOLWICH (*London*). — Concrete *Houses.*—The B.C. proposes to build 500 concrete houses at Eltham.

WOOLWICH (*London*).—The B.C. proposes to concrete ditches at Plumstead Marshes.

TENDERS ACCEPTED.

ACTON (*near Wrexham*).—*Sewage Works.*—The Wrexham R.D.C. has accepted the tender of Mr. J. Johnson, Rossett, at £1,880, for the construction of sewage disposal works at Acton.

BECKENHAM (*Kent*). — *Paving.* — The U.D.C. has accepted the tender of the Patent Victoria Stone Co., Ltd., at £1,815 5*s.* 6*d.* for the supply of about 5,000 super yds. of artificial stone paving.

CHESHUNT.—*Sewer, etc.*—The U.D.C. has accepted the tender of Messrs. W. and C. French, Buckhurst Hill, at £3,383 16*s.* for the construction of about 535 yds. of 24-in. diameter concrete sewer pipe.

CHESTERFIELD.—*Sewage Works.*—The B.C. has accepted the tender of Messrs. W. Moss & Sons, Loughborough, at £64,000, for the construction of sewage disposal works.

DEPWADE. — *Concrete Houses.* — The District Council has accepted a tender at £1,920, for the erection of four concrete houses.

DUNDEE. — *Foundation Work.* — The T.C. has accepted the tender of the Yorkshire Hennebique Contracting Co., Ltd., at £2,336, for reinforced concrete piling and foundation for the water supply extension.

DURHAM.—*Wall.*—The City Council has accepted the tender of Messrs. Holloway & Co., London, for the construction of a reinforced concrete wall.

GLASGOW. — *Floors.* — The T.C. has accepted the tender of Messrs. Melville Dundas & Whitson, at £6,954 7*s.* for the construction of reinforced floors for the new buildings of Stockwell Street Improvement scheme.

LEICESTER.—*Bridges.*—The T.C. has accepted the tender of Messrs. J. J. Shardlow & Co., Leicester, at £8,274 10*s.* for the construction of four concrete bridges.

LIVERPOOL. — *Concrete Houses.* — The T.C. has accepted the tender of Capt. R. B. Tinsley, for the erection of 100 concrete houses.

LUPSET.—*Concrete Houses.*—The Wakefield Corporation Housing Committee has accepted a contract, at £1,800, for the erection of four concrete houses.

MANCHESTER.—*Coal Bunkers, etc.*—The Corporation Gas Committee has accepted the tender of the Fram Reinforced Concrete Co., Ltd., at £75,000, for the construction of reinforced concrete coal bunkers, retaining walls, abutments and bridges, etc.

NORTHAMPTON. — *Concrete Houses.* — The T.C. has accepted the tender of Mr. C. J. Fox, Northampton, at £960, for the construction of a pair of concrete houses.

SOUTHEND-ON-SEA. — *Sewer Tubes.* — The Corporation has accepted the tender of Messrs. Thorby & Matthews, Leigh-on-Sea, at £5,944 5*s.* 10*d.*, for the construction of concrete tube sewers.

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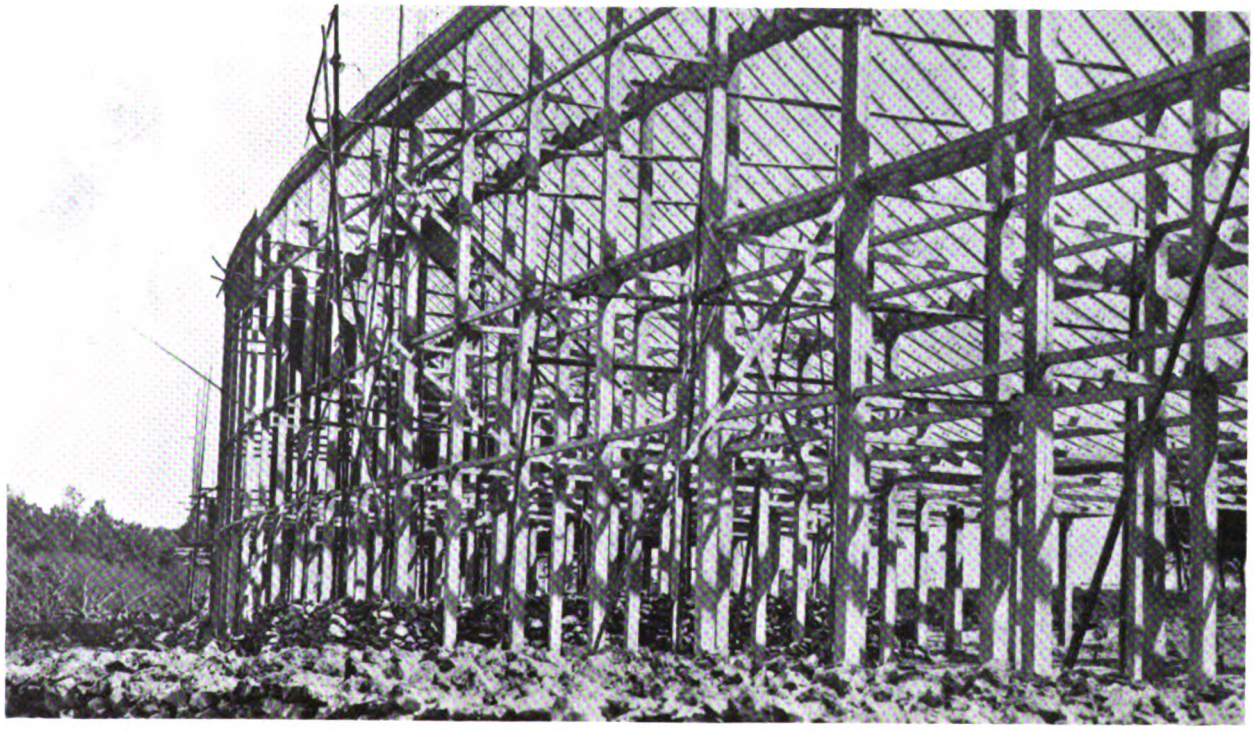

MOTOR RACE-TRACK AT LINAS-MONTLHERY: DURING CONSTRUCTION. (*See p.* 63.)

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume XX. No. 2 LONDON, FEBRUARY, 1925.

EDITORIAL NOTES.

Aluminous Cement.

THE article in this issue on aluminous cement is, we believe, the first to be published in this country dealing with the application of the principles of reinforced concrete construction to this new binding material. This article, together with those on rapid hardening Portland cement published this month and in our last issue, mark a stage in the development of the manufacture and application of cement, which has its significance in two directions. First it commences the abolition of the idea that there is only one sort of cement made and that it is for the users of cement to adapt their needs to the cement rather than for the manufacturer to adapt the cement to the needs of the user. It must be admitted that cement users have been wonderfully patient in accepting the limitations of cement which have existed until quite recently. It will appear almost incredible to later generations that while roads were being reconstructed, traffic in the streets of London was impeded to an extent that caused walking to be quicker than motoring because only one kind of cement was known and that took three weeks to harden. We are now at the beginning of an epoch when an engineer need only decide what period he can allow for concrete to harden to its maximum load-bearing capacity and the cement manufacturer will be able to supply his requirements; in this connection the term "to harden" means to acquire strength up to its full load-bearing capacity, plus the usual margin of safety. If a road engineer now wishes to open a road to traffic in twenty-four hours after laying the concrete he will order aluminous cement; if a day or two longer can be spared, then rapid-hardening Portland cement will be used, and if ten or fourteen days' delay can be permitted ordinary Portland cement will suffice. This is the position as it exists to-day, and there is good reason to believe that still further improvements will take place until the stage has been reached when a cement will be produced which will attain at any required period a strength at least equal to the aggregate to be mixed with it.

The second significance of this stage in the development of cement manufacture lies in the increased strength of the special cements now produced. Compression strengths in concrete of 1 : 2 : 4 proportions can now range up to 10,000 lbs. per square inch, and compared with this result it is obvious that the present basis of 600 lbs. permissible working stress is entirely out of proportion. A valuable opportunity for economy will be lost unless this basis can be adjusted to suit the altered circumstances. In this connection the proposed standard

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specification for concrete first issued in the United States in 1921 may point the way to a solution of the problem, because the specification put upon the contractor the onus of producing concrete of a minimum strength; and while this arrangement would leave the engineer free to design a structure according to the guaranteed strength of the concrete the field for economy by the use of special cements would undoubtedly be effectively explored by the contractor.

In a few words, the chief characteristic of aluminous cement is its property of rapid hardening. It will produce concrete which is harder at twenty-four hours than ordinary Portland cement concrete a month old. Aluminous cement was discovered in 1908, but was not freely marketed until some twelve years later. Until recently its manufacture was confined to France, but it is satisfactory to know that one of the British cement manufacturing firms has undertaken the production in the British Isles and is now able to supply aluminous cement upon a scale that is likely to satisfy all home demands. The period of seventeen years elapsing since the discovery of aluminous cement has been useful in establishing evidence as to stability and endurance with age. The "bogey" of failure in years to come should thus not arise in the minds of prospective users.

The essential constituent of aluminous cement is aluminate of lime, thus distinguishing it from Portland cement in which the principal constituent is silicate of lime. In principle the manufacture is very simple, amounting to nothing more than melting a mixture of alumina and lime and grinding the product to a fine powder. In practice, however, the manufacture is not an easy operation. A temperature of about 2,500 deg. F. must be attained; the molten material has a destructive action on the brickwork (even though refractory) with which it comes in contact; and the production of metallic iron (from impurities in the raw materials) which is frequently an accompaniment of the process adds complications to the grinding of the cement. The alumina for the manufacture is usually obtained in the form of the mineral known as bauxite, containing only about 60 per cent. of alumina, and as this is an imported mineral so far as England is concerned the cost of production of aluminous cement must necessarily be high. The orthodox furnace for the melting operation is a water-jacketed smelting furnace, but the electric furnace is also used and there is no doubt that other types of furnaces could be adapted for the purpose.

The early investigations in connection with the discovery of aluminous cement were conducted by Bied in France and by Spackman in the United States, each ignorant of the work of the other. It is curious that the object sought for was not a rapid-hardening cement in either case. Bied was seeking a cement that would not be subject to chemical decomposition by the sulphates in sea water and in certain soils, while Spackman's aim was to produce a material that would serve as an accelerant to the hardening of lime. Bied was successful in his search, and resistance of aluminous cement to the action of sea water is a claim as well established as that for the possession of rapid-hardening qualities. From a practical point of view this property of resistance to sea water means that whereas a Portland cement concrete must be made impermeable in order to resist the destructive action of sea water (not a difficult task to experienced concreters), an aluminous cement concrete is not subject to chemical disintegration even though porous.

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Aluminous cement, owing to its high cost of production, is hardly likely to become generally employed in constructional work, but experience has shown that there are cases where its use is justified and advantageous and it is not to be doubted that such cases will multiply, especially if the price be reduced as the result of longer experience with the manufacture.

Report on Concrete Cottages.

LOCAL authorities and others responsible for the erection of housing accommodation, as well as those who have for so long advocated the use of concrete as a material for cottage construction, will welcome the second interim report of the Committee appointed by the Ministry of Health to consider new methods of house construction. So many conflicting statements have been made as to the merits of different methods and materials, from new ways of laying bricks to steel and even corrugated iron, that it must be difficult indeed for a local council, mostly composed of laymen, to decide which to choose. Concrete has been adopted for so many thousands of houses, and when properly used has stood the test of time so well, that one would have thought there could have been no doubt as to its suitability and economy, but there are still some architects and builders who prefer to use time-honoured materials to which they and their employees are accustomed. The result of this policy is now apparent, and we see the small builder who is unwilling to adapt himself to the new conditions being superseded by large contracting firms who specialise in concrete cottage construction. The antagonism of the building trade unions to this material is also reflected in the development of methods of *in situ* construction which the bricklayer by no stretch of the imagination can claim to be his own particular birthright. We would advise all concerned with housing to consider carefully the recommendation of the report (given elsewhere in this issue) that "there should be no prejudice against a concrete building," for no prejudices or preferences, however strong, can stand in the way of such an urgent national necessity as the provision of more houses.

The first report of the Committee dealt with steel houses, and was so carefully worded and so full of reservations that it was obvious the members were not favourably impressed with the suitability of this material for cottage construction. Moreover, we find that the estimated cost of a steel house is no lower than recent tenders for concrete, such as the contract entered into by the Birmingham Corporation for the erection of 1,300 concrete cottages at £425 each, and from what has happened at Glasgow it seems probable that there will be serious labour difficulties to overcome before they can be built at all. The report on concrete, however, can only be described as wholeheartedly favourable—even enthusiastic. This Committee of architects, engineers, builders, and operatives tells us that "houses built of concrete are satisfactory habitations"; that "this method of building enables houses to be completed rapidly and almost entirely by other than skilled labour"; that "this method of construction promises more houses and more permanent structures than any other alternative to brick"; and that "the maintenance costs of such houses will compare favourably with those of any other houses." In this emphatically favourable report of a committee of experts local authorities and others contemplating building

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schemes now have something definite to work upon and will no doubt follow its advice and resolve their difficulties by the erection of houses which, in the words of the Committee, form "a satisfactory and comfortable dwelling, satisfactory in appearance, built of materials found ready to hand in most districts, and which require no skilled labour."

The report gives some useful notes on the relative merits of the different methods of using concrete, and emphasises the point which we have often made that no house, whether built of concrete or any other material, can be satisfactory unless the preparation of the materials and the work of erection are thoroughly supervised and properly carried out.

Cement and Heat Insulation.

THE growing development of cold storage for food has led to the scientific study of this subject from a constructional standpoint, and some valuable data upon the heat insulating properties of various materials are given in a recent report of the Department of Scientific and Industrial Research.* The best heat insulator is a vacuum (e.g. the vacuum flask), and still air is probably the next best. Hence the materials generally employed as heat insulators are those, such as cork, charcoal, and slag wool, having a cellular structure in which a minimum amount of solid material is combined with a maximum volume of air divided into minute spaces. From the weights per cubic foot of the insulating materials used in the tests described in the report it can be calculated that there is as little as 10 per cent. of solid material in many insulating blocks, the remainder being air in numberless voids. This being so, it is evident that the heat conducting property of the solid material itself is of minor importance compared with its property of air occlusion. For example, solid slag conducts heat readily, but slag in the fibrous form (slag wool) is an excellent heat insulator because it has 90 per cent. of voids.

Bearing these facts in mind it is to be expected that ordinary concrete (which when properly made is a dense material) has little value as a heat insulator, but it is interesting to observe that cement itself is a much better insulator than sand or stone. The flow of heat through a slab of neat cement would be only one quarter of that passing through a concrete slab, and only a tenth or even less of that passing through a slab of solid stone. Hence neat cement appears to be a suitable basis for a heat insulating material provided it can be worked up in a manner to produce a porous block with a large proportion of voids. In this connection, particulars of various patent processes have been published which suggest that this is possible, and in course of time an important addition may be made to the industrial applications of cement.

* First Report on Heat Insulators. (Special Report No. 5 by the Engineering Committee of the Food Investigation Board of the Department of Scientific and Industrial Research. Price 2s. net.)

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NEW MOTOR RACE-TRACKS IN FRANCE.

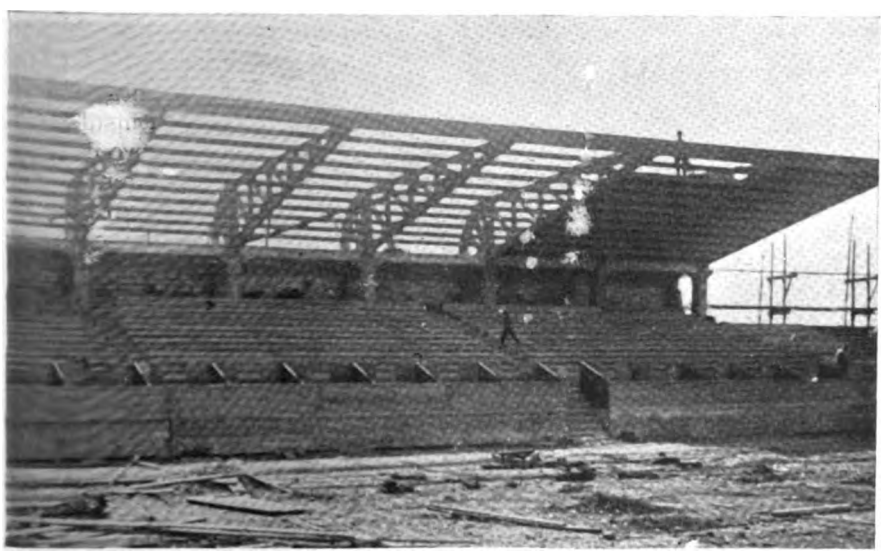
Track at Miramas, near Marseilles.

In spite of the extensive progress achieved by the French motor-car industry there has hitherto been no track on which racing contests could be decided. In July last, however, an "autodrome" was opened at Miramas (Bouches du Rhône), about twenty-five miles from Marseilles, and another now opened at Montlhery, near Paris.

The Miramas track is oval in shape, about 5,468 yds. long by 17½ yds. wide. The Brooklands track, it may be mentioned, is 4,630 yds. long and 33 yds. wide. Along the straight sides the track is inclined at a slope of 1 in 36, while in the curved parts the slope is increased to 1 in 20. The central 17½ yds. of the track used for racing is of concrete on a macadam foundation, and on each side of this are other tracks for the circulation of cars, making the total width 27 yds. Within the space enclosed by the track are smaller tracks and courses for steering and other tests.

The average thickness of the concrete of the main track is 6 inches. The concrete was mixed in a 1⅓ cu. yd. capacity mixer mounted on a bridge travelling on a light railway at each side of the track, and was turned out at the rate of 26 cu. yds. per hour. After being placed in position through a chute it was roughly levelled and then tamped and screeded by template and belt. No expansion joints have been provided.

The stand is built entirely of reinforced concrete, to the design of Monsieur H. Lossier. Its total length is 984 ft. and its width 41 ft. There are twelve rows of terracing steps and a row of boxes along the front, providing accommodation for 7,000 spectators. The roof trusses rise from a reinforced



COVERED STAND AT THE NEW MOTOR RACE-TRACK AT MIRAMAS.

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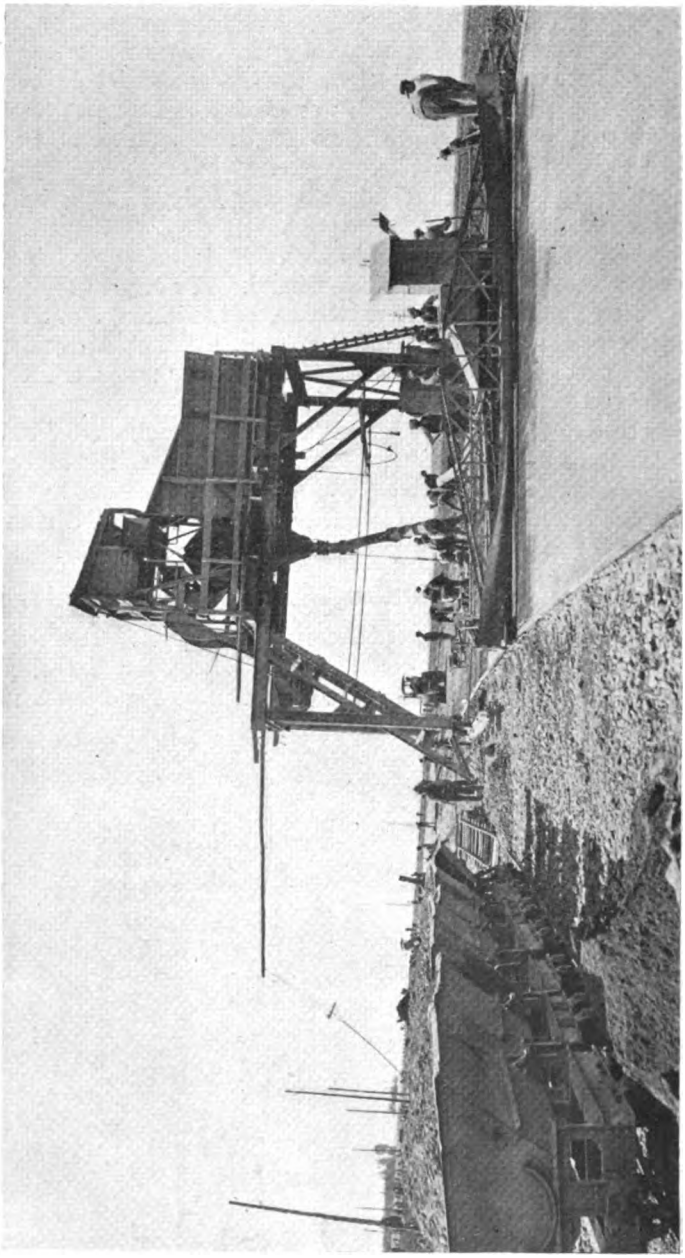

NEW MOTOR RACE-TRACK AT MIRAMAS: VIEW DURING CONSTRUCTION.

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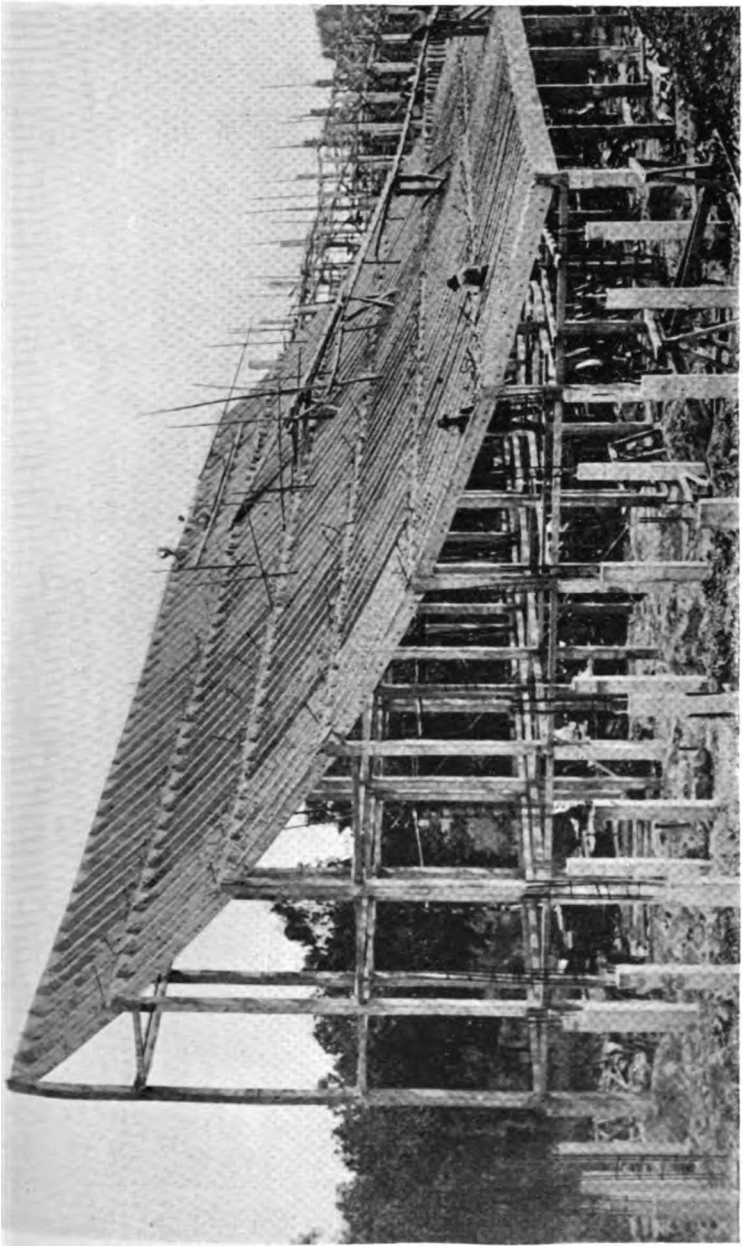

MOTOR RACE-TRACK AT LINAS-MONTLHERY: DURING CONSTRUCTION.

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FRAMEWORK DURING CONSTRUCTION.

FRAMEWORK COMPLETED READY FOR PLACING OF FLOOR.

MOTOR RACE-TRACK AT LINAS-MONTLHERY.

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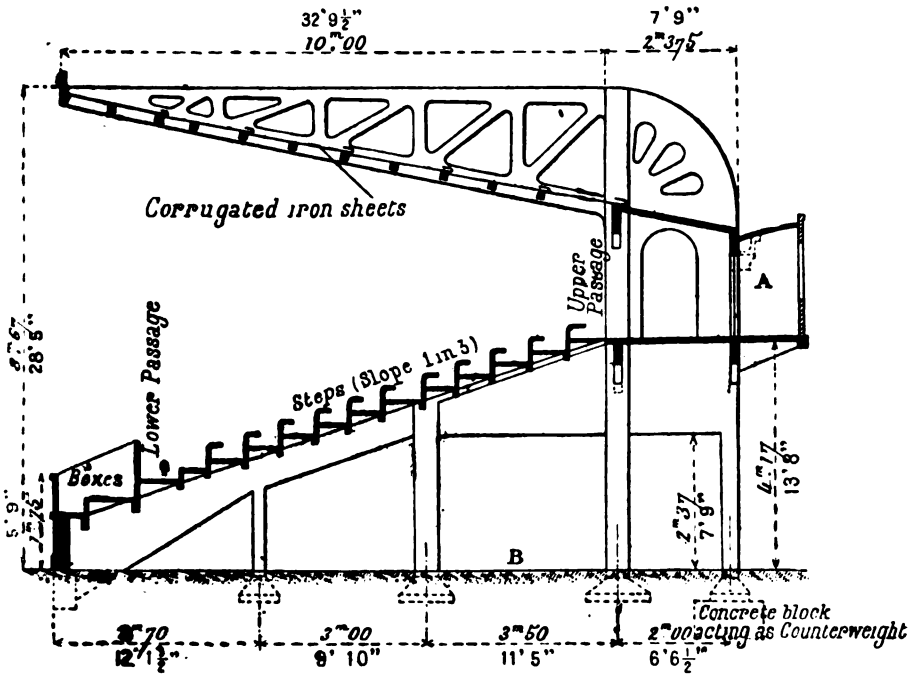

["A," Wireless Station; "B," Workshops, Cloakrooms, Attendants, etc.]

NEW MOTOR RACE-TRACK AT MIRAMAS: TRANSVERSE SECTION OF COVERED STAND.

concrete framework resting on a mass-concrete slab which acts as a counterweight to equalise the turning moment due to the weight of the roof. The light reinforced concrete beams resting on the roof trusses are covered with corrugated iron sheets.

The autodrome is completely enclosed by an encircling reinforced concrete wall 8,200 yards in length.

Track at Linas-Montlhery.

THE track at Linas-Montlhery is of special interest in that it is erected on reinforced columns, and also because of the extensive use of pre-cast units. The track is oval shape, with a total track length of 2,734 yards. The width of the track is 59 ft. according to the horizontal projection, but as the outer edge is 34 ft. higher than ground level the actual distance along the line of curvature between the edges is nearly 70 ft.

The track is completely built up on a framework of reinforced concrete on reinforced concrete columns of a height according to the contour of the ground in relation to the curvature of the surface of the track. There are over 3,300 columns, about 18,000 pre-cast beams, and 7,000 longitudinal tie-beams connecting the tops of the columns. In order to simplify erection no diagonal beams are used, the anchorage of the framework to a heavy retaining wall at its inner edge being relied upon to counteract wind pressure. The columns and beams were

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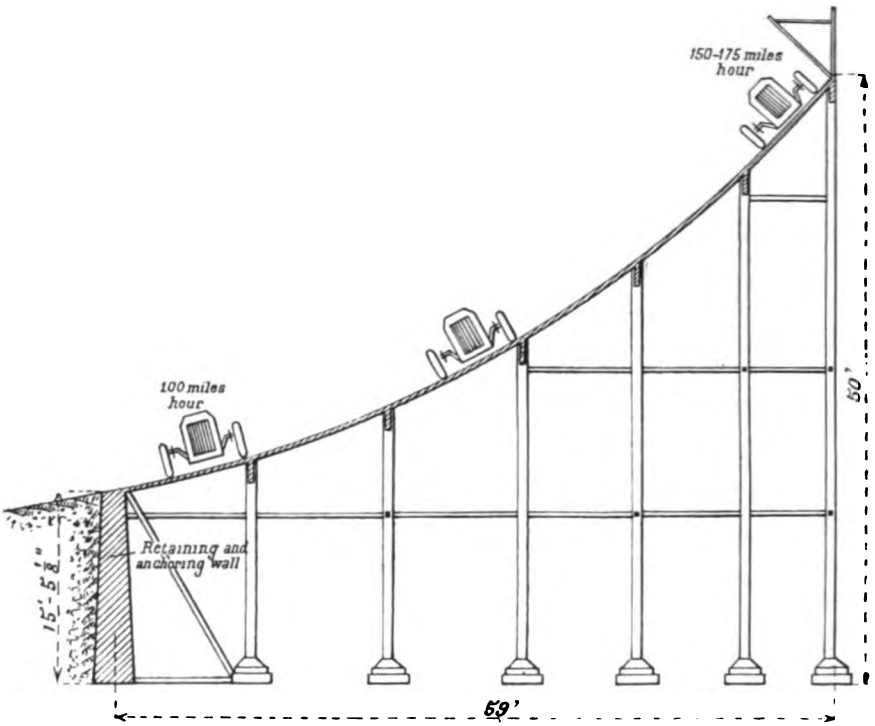

MOTOR RACE-TRACK AT LINAS-MONTLHERY SECTION.

pre-cast on the site, and were made and placed in position with such accuracy that nowhere has the level of the track varied by more than one-fifth of an inch. Speed of erection was essential, and as a matter of fact the whole of the work was completed within four months. In filling in the track between the framework wooden formwork was suspended by hooks from iron rings cast into the beams.

The columns and tie-beams were then erected, and grouted together. Shuttering for the main longitudinal beams, which were cast *in situ*, was then suspended by hooks from rings cast into the pillars, and the pre-cast cross-beams placed in position. The next step was to pour the monolithic main beams. Shuttering for the floor was next suspended from rings cast in the cross girders, and after the reinforcement was placed in position the floor was poured.

The floor is 4¾ in. thick, and the surface left rough without any surface rendering, as it is anticipated that an untreated surface will absorb oil and grease from the cars. Expansion joints are provided at regular intervals. Owing to the speed with which the work had to be completed wood was made use of for the stand, but a reinforced concrete structure is to be erected later.

The track was designed by Mr. Jamin, and erected by the Société Nationale de Construction.

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"THE CONCRETE YEAR BOOK, 1925."

THE second edition of "The Concrete Year Book" is now ready, and in view of the great demand orders should be placed at once to make sure of securing a copy.

The new edition, which is edited by Dr. Oscar Faber, O.B.E., D.Sc., M.Inst.C.E., and the Editor of Concrete Publications, Limited, is on substantially the same lines as last year, that is, divided into three sections containing respectively a Handbook, a Directory, and a Catalogue.

A certain amount of the Handbook Section is retained, after thorough revision, and many new chapters have been added. It is the intention to give in this section the results of all important researches carried out during the previous year so that they will be available to subscribers in handy form and also to give authoritative articles on difficult aspects of concrete and reinforced concrete work and the latest developments in the manufacture and use of materials and in concrete construction. The chapters include "Cement"—in which rapid-hardening Portland cement, aluminous cement, coloured cement, hot cement, the storage of cement, etc., are dealt with; "Essential Factors in Making Good Concrete"—including tests for cement, quality of aggregates, water content, workmanship, setting time, density, rodding, the slump test, etc.; "Quantity of Water required for producing the Best Concrete"; "Quality of Water for Concrete"; "Tables for Reinforced Concrete Design"; "Concrete Floors"—in which designs are given for different types of concrete floors; "Concrete Sewers and Conduits"—in which every type of concrete sewer is illustrated and described; "Concrete Roads"—containing a specification for all-concrete roads, with notes on cost, materials, etc.; "Concrete Cottages"—giving elevations, plans and sections of a concrete cottage with a full bill of quantities; "Surface Treatment of Concrete"—with notes on exposed aggregate, coloured cement, colouring materials, rendering, American methods, cement washes, rubbed finishes, tooled finishes, etc.; "Fire Resistance of Various Aggregates." There are also chapters containing a great deal of memoranda of every-day use to designers and contractors, and a complete bibliography of every book published in the English language dealing with concrete, reinforced concrete, and allied subjects.

This year the Directory Section is much larger, and contains several new features, including a complete list of special trade names and brands in use in the concrete industry with the names and addresses of their proprietors.

The important Catalogue Section, which is complementary to the Directory Section, is also much larger this year. Its object is to give firms in the concrete industry, or catering for the concrete industry, an opportunity of describing their businesses or products in more detail than is possible in the Directory, and within the 180 pages of this section a very large number of machines, products, and businesses are described, with copious illustrations; this section should be invaluable to those seeking firms of contractors to undertake special kinds of work or a machine or product for a special purpose.

The book comprises 400 pages, and is sold at 3*s*. post free.

The first edition of this annual, issued on January 1 last year, was accorded an excellent reception by the press, and we have received many letters from purchasers expressing their appreciation, but perhaps the best indication of the value of the volume is the fact that the whole of the first edition, of 3,000 copies, was exhausted before the end of the year.

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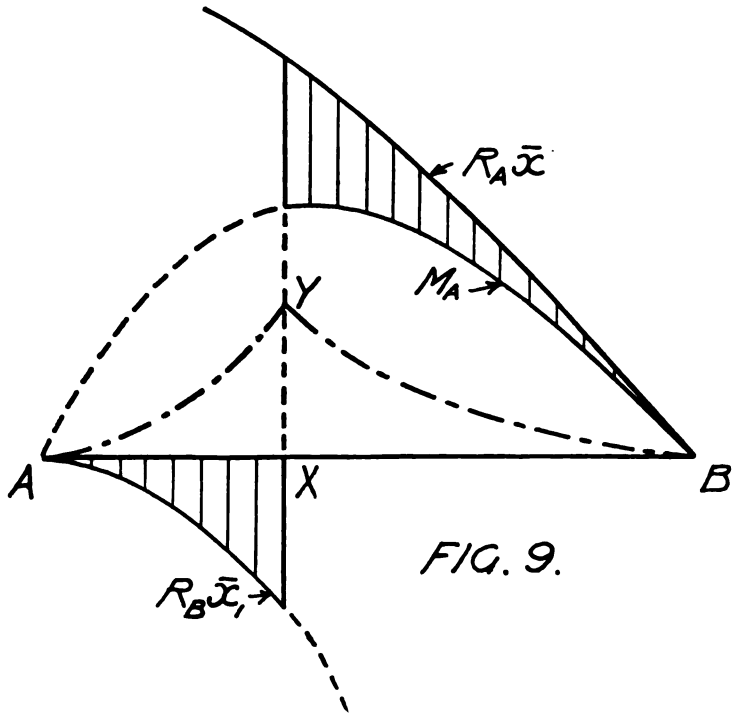

FIG. 9.

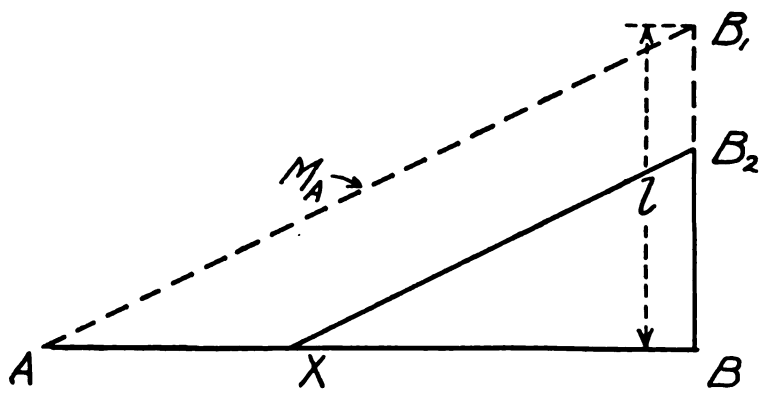

FIG 10

INFLUENCE LINE DIAGRAMS. (*See p.* 71.)

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INFLUENCE LINE DIAGRAMS FOR SHEARING FORCE, BENDING MOMENT, SLOPE, AND DEFLECTION FOR SINGLE-SPAN GIRDERS. II.*

By W. NORMAN THOMAS, M.A., D.Phil., M.Sc., A.M.Inst.C.E.

(c) Beam Encastered at A and Supported at B.

In this case the fixing moment $M_A = -\frac{ab\,(a+2b)}{2l^2}$ or $\frac{b^3}{2l^2} - \frac{b}{2}$, so that the influence line for M_A is independent of the position of X and is as shown in *Fig.* 9. The maximum value of $M_A = 0{\cdot}192\ Wl$ when $b = l/\sqrt{3}$.

The bending moment at X is

$$M = R_A\bar{x} + M_A \text{ when } a > \bar{x}$$
$$\text{and } M = R_B\bar{x}_1 \quad ,, \quad a < \bar{x}$$

The influence lines for R_A and R_B have already been drawn in *Fig.* 4, so that the influence lines for M are now easily drawn as in *Fig.* 9.

As for the "fixed" beam, the curve M_A is constant for all positions of X, while the heights of the curves representing $R_A\bar{x}$ and $R_b\bar{x}_1$ are dependent upon the values of $\bar{x}$ and $\bar{x}_1$.

The influence line reduced to a horizontal base is given by the curves AYB.

When $\bar{x} > l/3$ as in *Fig.* 9, the bending moment at X is positive for all positions of the load; but when $x < l/3$ the bending moment at X will be negative if $b^2 < l^2(l - 3\bar{x})/(l - \bar{x})$.

Whatever be the loading on the beam therefore, the point of contraflexure will occur at a position less than $l/3$ from the fixed end A.

(d) Beam Encastered at A and Free at B.

In this case the influence line diagram for bending moment is very simple. As $M_A = -a$, the influence line for M_A is AB_1B (*Fig.* 10) where BB_1 represents l to any convenient scale. Thus the value of $M_A = -Wl$ when a concentrated load W is situated at the free end B of the cantilever.

The influence line for bending moment at X is represented by AXB_2B where XB_2 is parallel to AB_1. The sign of the bending moment is always negative.

Influence lines for various positions of X can obviously be all placed upon the same diagram.

INFLUENCE LINES FOR SLOPE.

(a) Simply-Supported Beam.

The slope at any point X in the beam is given by equations (3) and (3a), i.e.,

$$\frac{dy}{dx} = \frac{1}{EI}\left(R_A\frac{\bar{x}^2}{2} + K_1\right) \text{ when } a > \bar{x}$$

or

$$\frac{dy}{dx} = \frac{1}{EI}\left(R_B\frac{\bar{x}_1^2}{2} + K_2\right) \quad ,, \quad a < \bar{x}$$

[* Concluded from November, 1924.]

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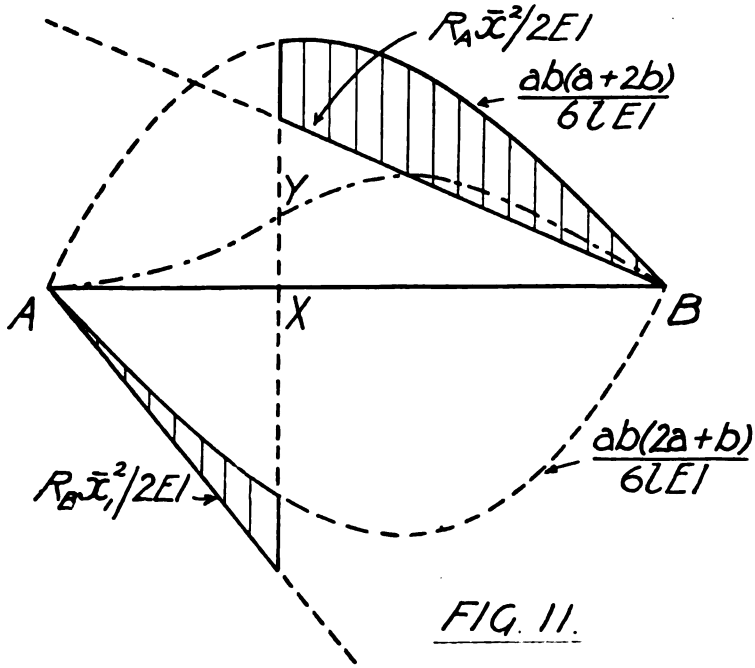

FIG. 11.

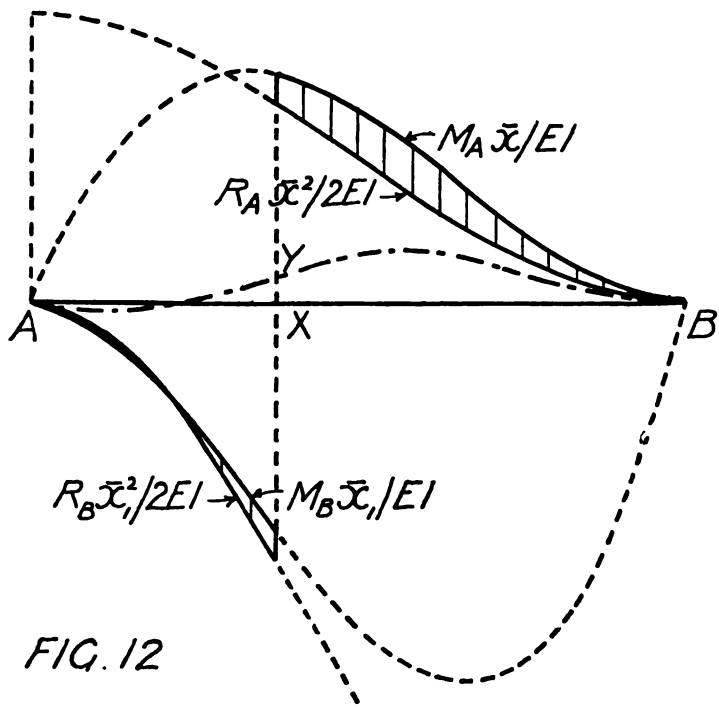

FIG. 12

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We have shown that $K_1 = -\dfrac{ab(a+2b)}{6l}$ and $K_3 = -\dfrac{ab(2a+b)}{6l}$, while the influence lines for R_A and R_B are given in *Fig.* 2. The influence line diagram for slope may therefore be derived as previously explained: it is represented by the shaded areas in *Fig.* 11. The curve AYB shows the intercepts calculated and plotted directly from the horizontal base AB, when $x = \frac{3}{8}l$.

For the sign of the slope to change while $a > \bar{x}$, $R_A\dfrac{\bar{x}^2}{2}$ must become numerically equal to K_1.

In such a case

$$\frac{b}{l}\frac{\bar{x}^2}{2} = \frac{ab(a+2b)}{6l}$$

i.e.

$$3\bar{x}^2 = a(a+2b)$$

or

$$\bar{x}^2 = (l^2 - b^2)/3$$

The conditions state, however, that the greatest value b can have is $(l - \bar{x})$, and the least value is 0. Substituting these values for b, it is seen that $\bar{x}$ must lie between the limits $l/2$ and $l/\sqrt{3}$. Similarly for the sign of the slope to change while $a < \bar{x}$, $\bar{x}_1$ must lie between the limits $l/2$ and $l/\sqrt{3}$.

The slope at any particular point X thus remains of the same sign for all positions of the load, unless X is nearly in the centre of the span, i.e. is between $l/2$ and $l/\sqrt{3}$ from either end.

The position at which the maximum deflection of a beam occurs is that at which the slope of the beam is zero. This point therefore must lie within the limits of $0{\cdot}423l$ and $0{\cdot}577l$ from either support.

The rule applies not only to the case of a single concentrated load, but, as is obvious from the general properties of influence line diagrams, it applies to any form of direct loading upon a simply-supported beam.

(*b*) Beam Encastered at A and B.

The slope at any point X in the beam is given by equations (3) and (3*a*), i.e.,

$$\frac{dy}{dx} = \frac{1}{EI}\left(R_A\frac{\bar{x}^2}{2} + M_A\bar{x}\right) \text{ when } a > \bar{x}$$

and

$$\frac{dy}{dx} = \frac{1}{EI}\left(R_B\frac{\bar{x}_1^2}{2} + M_B\bar{x}_1\right) \quad ,, \quad a < \bar{x}$$

The influence lines for R_A, R_B, M_A and M_B have already been drawn in *Figs.* 3 and 7, and the quantities $\bar{x}$ and $\bar{x}_1$ are constants for any particular position of X. E and I are also constants.

The influence line diagram is therefore represented by the shaded areas in *Fig.* 12, while the curve AYB shows the influence line calculated and plotted directly from the horizontal base AB, for $x = 0{\cdot}375l$.

In this case, the slope at any particular point X remains of the same sign for all positions of the load, unless X lies within the middle third length of the beam. When X lies within this middle third length, the slope at X is of varying

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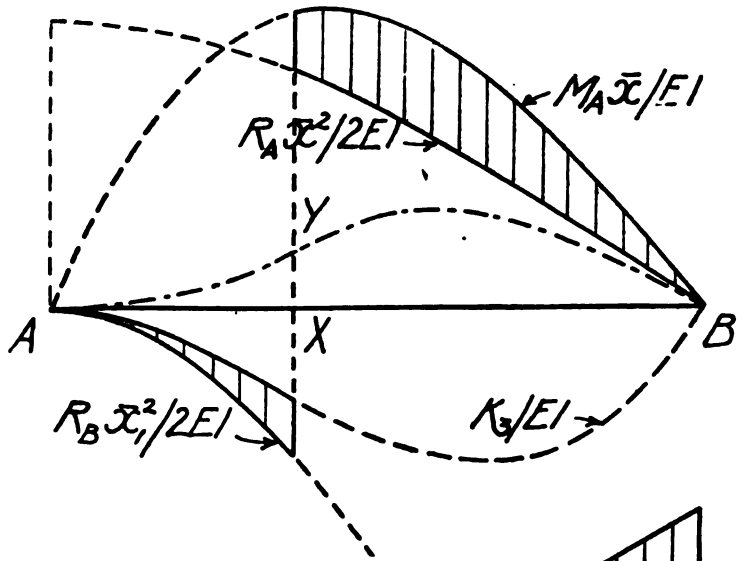

FIG. 13.

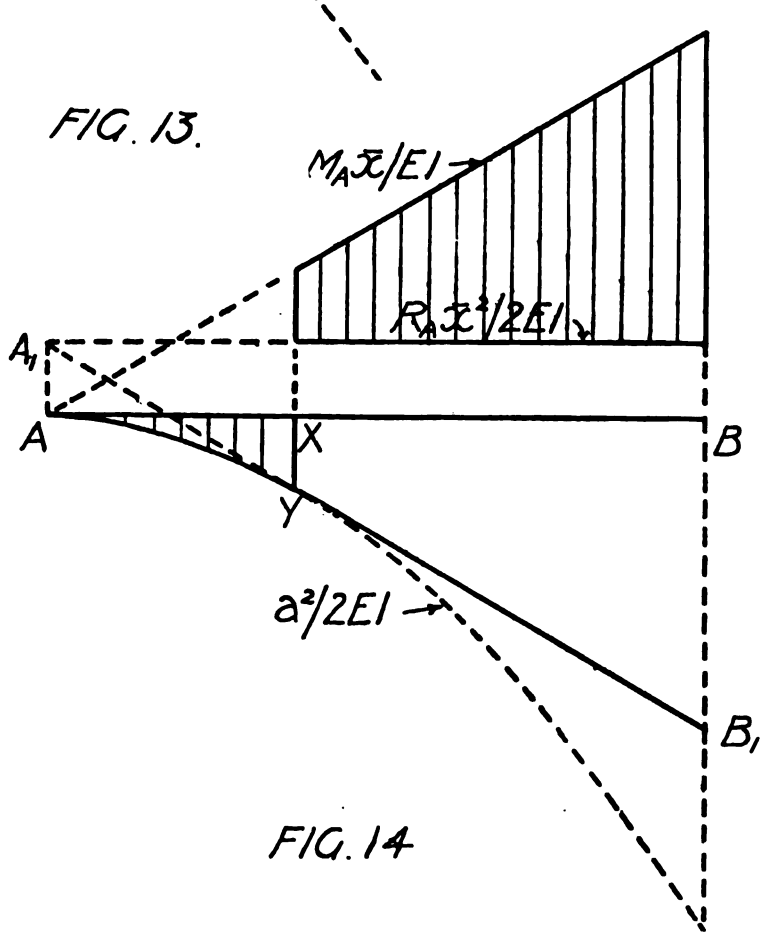

FIG. 14

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sign according to the position of the load. It follows then, that for any loading whatever on a "fixed" beam, the change of slope, and therefore the maximum deflection of the beam, must occur within these limits.

(c) Beam Encastered at A and Supported at B.

In this case the equations for slope are

$$\frac{dy}{dx} = \frac{1}{EI}\left(R_A\frac{\bar{x}^2}{2} + M_A\bar{x}\right) \text{ when } a > \bar{x}$$

and

$$\frac{dy}{dx} = \frac{1}{EI}\left(R_B\frac{\bar{x}_1^2}{2} + K_3\right) \quad ,, \quad a < \bar{x}$$

The value of $K_3 = -\frac{a^2b}{4l}$, which is independent of $\bar{x}$, and the curve for K_3/EI is shown in *Fig.* 13.

The influence line diagram for slope may therefore be drawn as in the previous examples (see *Fig.* 13), using the influence lines for R_A, R_B, M_A and K_3, and the particular values of $\bar{x}$ and $\bar{x}_1$ at the point X. *Fig.* 13 is drawn for a value of $\bar{x} = 0{\cdot}375l$. In this case (c) X must lie in such a position that $\bar{x} > 0{\cdot}423l$ and $< 0{\cdot}667l$ if there is to be a change in the sign of the slope for various positions of the load.

If any loading whatever is applied to such a propped cantilever, the maximum deflection must occur at some point between these limits.

(d) Beam Encastered at A and Free at B.

For a cantilever the equations for slope are

$$\frac{dy}{dx} = \frac{1}{EI}\left(R_A\frac{\bar{x}^2}{2} + M_A\bar{x}\right) \text{ when } a > \bar{x}$$

and

$$\frac{dy}{dx} = \frac{1}{EI}K_3 \quad ,, \quad a < \bar{x}$$

while $R_A = 1$, $M_A = -a$, and $K_3 = a^2/2$.

The influence line diagram is shown by the shaded areas in *Fig.* 14. Alternatively to draw the complete diagram upon a horizontal base, construct the parabola $a^2/2EI$ from $a = 0$ to $a = l$; at X erect an ordinate to cut this curve at Y; at A set off $AA_1 = \bar{x}^2/2EI$; join A_1Y and produce to B_1. The influence line is then AYB_1B. Several influence lines for various positions of X can thus be drawn upon the same diagram.

INFLUENCE LINES FOR DEFLECTION.

(a) Simply-Supported Beam.

The equations for deflection are obtained from equations (4) and (4a), i.e.,

$$y = \frac{1}{EI}\left(R_A\frac{\bar{x}^3}{6} + K_1\bar{x}\right) \quad \text{when } a > \bar{x}$$

and

$$y = \frac{1}{EI}\left(R_B\frac{\bar{x}_1^3}{6} + K_3\bar{x}_1\right) \quad ,, \quad a < \bar{x}$$

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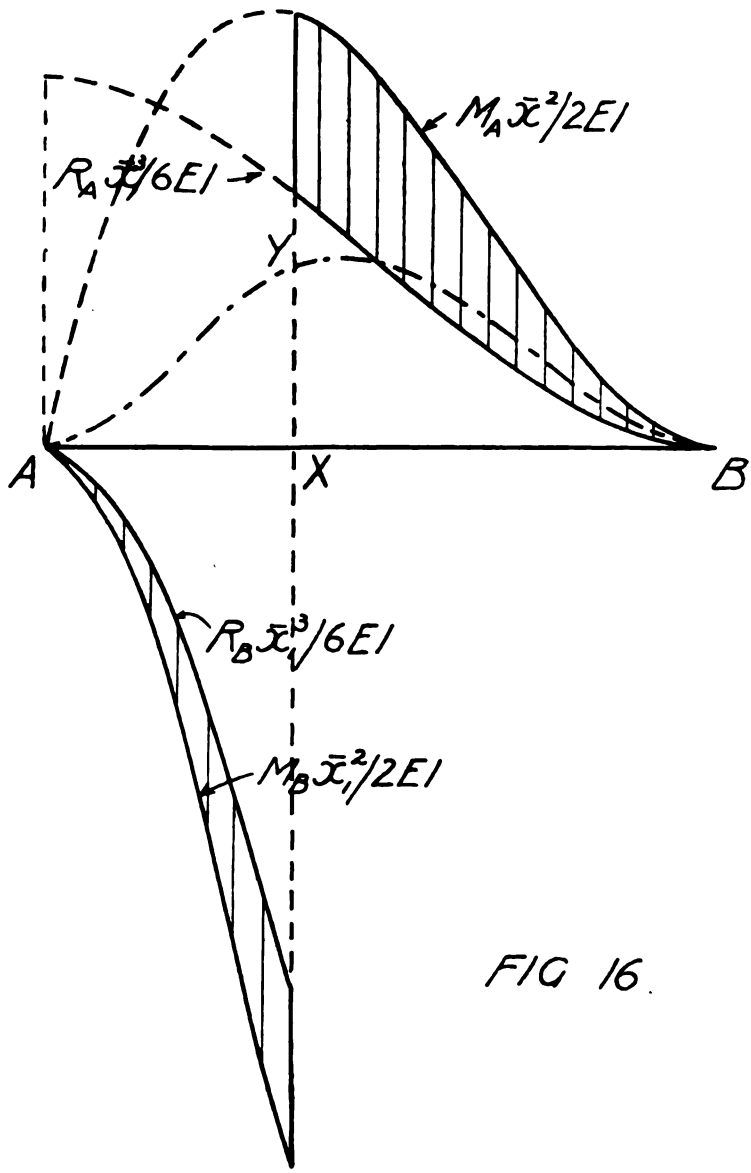

FIG 16.

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The influence lines for R_A, R_B, K_1 and K_3 have already been drawn, so that treating $\bar{x}^3/6$, $\bar{x}_1^3/6$, $\bar{x}$ and $\bar{x}_1$ as constants, *Fig.* 15 is easily obtained, the shaded areas, as before, representing the influence line diagram. The influence line AYB for the deflection at X, is also shown, the ordinates being plotted from the horizontal base line AB.

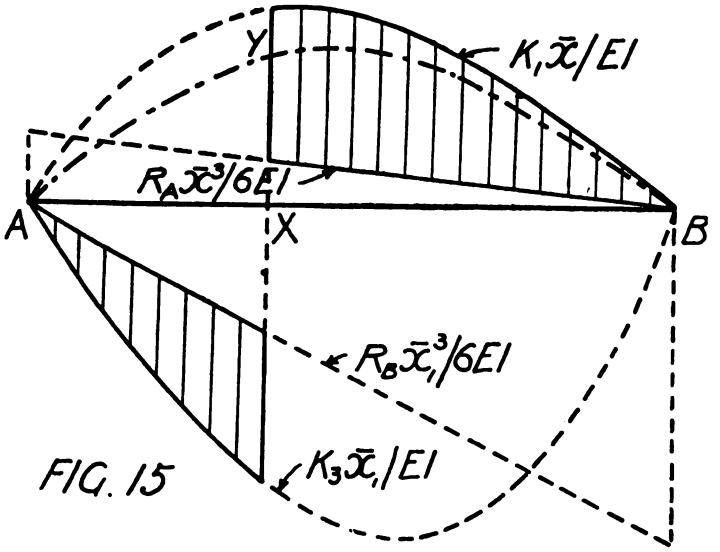

FIG. 15

(*b*) Beam Encastered at A and B.

In this case the equations for deflection become

$$y = \frac{1}{EI}\left(R_A\frac{\bar{x}^3}{6} + M_A\frac{\bar{x}^2}{2}\right) \quad \text{when } a > \bar{x}$$

and

$$y = \frac{1}{EI}\left(R_B\frac{\bar{x}_1^3}{6} + M_B\frac{\bar{x}_1^2}{2}\right) \quad \text{,,} \quad a < \bar{x}$$

From the influence lines for R_A, R_B, M_A and M_B the influence line diagram for the deflection at X is determined as before, and is shown in *Fig.* 16.

(*c*) Beam Encastered at A and Supported at B.

The equations for deflection in this case are

$$y = \frac{1}{EI}\left(R_A\frac{\bar{x}^3}{6} + M_A\frac{\bar{x}^2}{2}\right) \quad \text{when } a > \bar{x}$$

and

$$y = \frac{1}{EI}\left(R_B\frac{\bar{x}_1^3}{6} + K_3\bar{x}_1\right) \quad \text{,,} \quad a < \bar{x}$$

The influence line diagram for the deflection at X is shown in *Fig.* 17, being derived from the previously drawn influence lines for R_A, R_B, M_A and K_3, the functions of $\bar{x}$ and of $\bar{x}_1$ being constants for any particular values of X.

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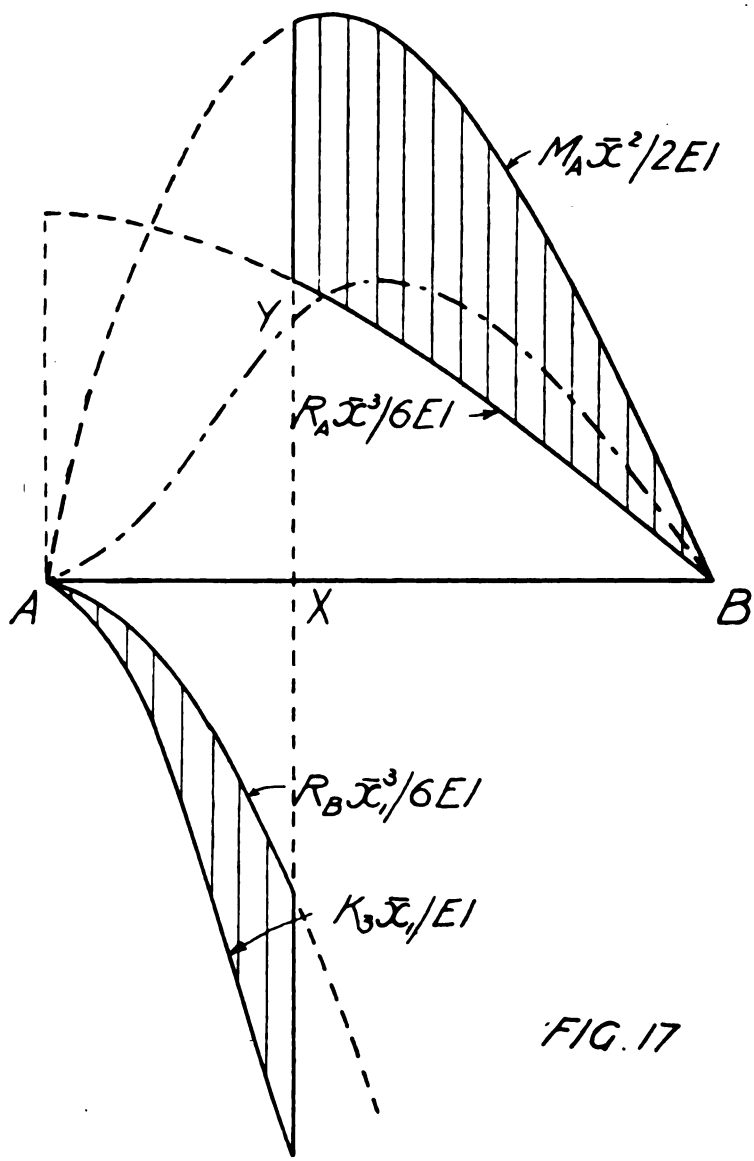

FIG. 17

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FIG. 18

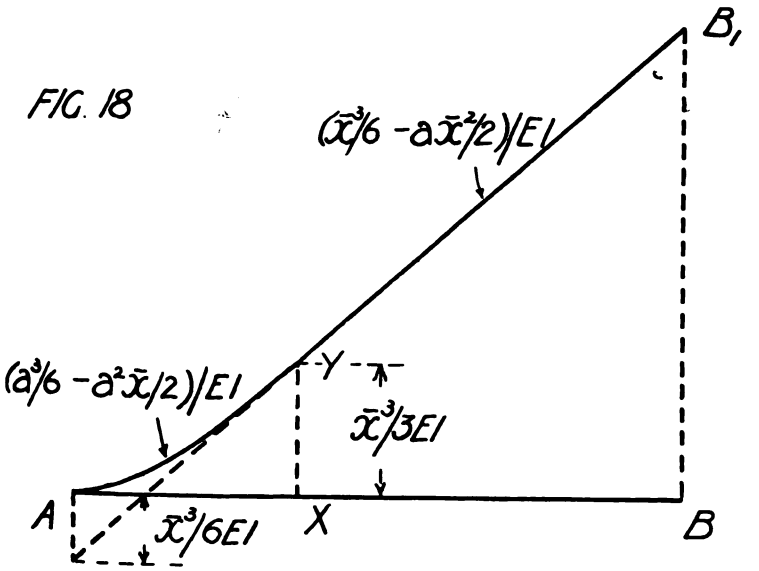
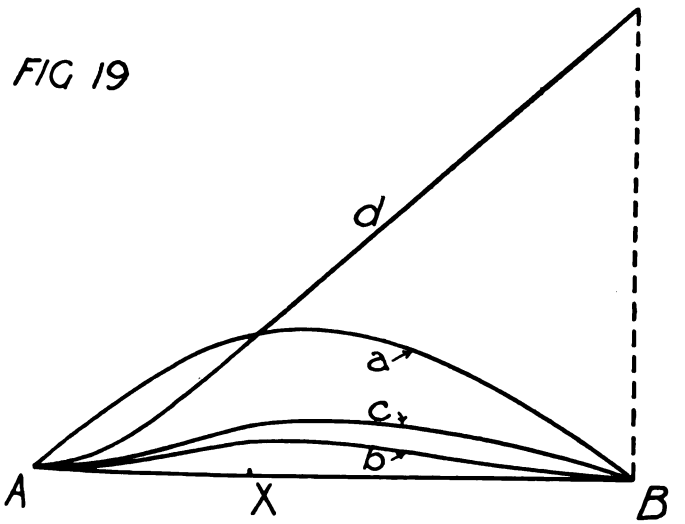


FIG 19



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(*d*) Beam Encastered at A and Free at B.

The equations for deflection, obtained from equations (4) and (4*a*), are

$$y = \frac{1}{EI}\left(R_A\frac{\bar{x}^3}{6} + M_A\frac{\bar{x}^2}{2}\right) \text{ when } a > \bar{x}$$

$$y = \frac{1}{EI}\ K_3\bar{x}_1 + K_4 \qquad ,, \quad a < \bar{x}$$

As before, $R_A = 1$, $M_A = -a$, $K_3 = \frac{a^2}{2}$, $K_4 = \frac{a^3}{6} - \frac{a^2 l}{2}$, so that

$$K_3\bar{x}_1 + K_4 = \frac{a^3}{6} - \frac{a^2\bar{x}}{2}$$

The influence line for the deflection at X is shown in *Fig.* 18, being linear from $a = \bar{x}$ to $a = l$.

Fig. 19 has been drawn on the assumption that the product EI is the same in each of the four cases that have been considered; it shows for the purposes of comparison the four influence lines for deflection drawn to the same scale.

BOOK REVIEW.

Structural Design in Steel Frame Buildings. By Percy J. Waldram.

(London: B. T. Batsford, Ltd. Pp. 73. Price 12s. 6*d*. net.)

THIS volume sets out the design of six large typical structural members in steelwork, taken from actual practice, which are fully calculated and detailed in accordance with the Act which regulates in London the design of steel frame buildings with thin walls. The primary object aimed at in the selection of the various examples has been the solution of those problems which frequently arise in design under the Act. The details in some cases have also purposely been varied in order to incorporate a number of minor problems, and to include within a necessarily small compass a fairly comprehensive survey of design in structural steelwork as applied to ordinary buildings. The structures dealt with include a typical large compound girder; a riveted plate girder; a typical stanchion carrying eccentric loads; grillage foundations; wall stanchion carrying main floor girder; steel mansard roof rib. There is a brief introduction relating to the practice of steel frame buildings in this country, and a full specification of each example of which the text consists; the design of each feature and detail is considered and worked out and illustrated by diagrams and comparative tabulated figures. A section on tables contains pages of the geometric and structural properties of British Standard beams and channels; reductions of moment of inertia due to rivet holes in various structural members; strength and bearing area of rivets and bolts, and other formulæ. Other appendices deal with approximate estimates of steelwork; estimation of weight of parallel plate girders; deflection of beams due to unsymmetrical loadings, and the L.C.C. Regulations for Structural Steelwork.

Books Received.

Detailing and Fabricating Structural Steel. By F. W. Dencer.

(London: McGraw-Hill Publishing Co., Ltd. Pp. 511; illus. 328. Price 25s. net.)

Structural Engineering: Fundamental Properties of Materials. By G. F. Swain.

(London: McGraw-Hill Publishing Co., Ltd. Pp. 200; illus. 49. Price 12s. 6*d*. net.)

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CONCRETE COTTAGES FOR HOUSING SCHEMES.

Favourable Report of the Ministry of Health Committee.

In its second interim report, referred to in our Editorial pages, the Committee set up by the Ministry of Health to consider new methods and materials of construction available for building working-class houses deals solely with concrete, which is strongly recommended as an "effective supplement to the normal methods of building."

The report contains the following:

Introduction.

It is the considered opinion of the Committee that, broadly speaking, the various forms of concrete construction offer a very effective supplement to the normal methods of building. Not only is the house built in concrete satisfactory, but it utilises labour and material in plentiful supply. The Committee, therefore, recommend that local authorities should, wherever necessary, include in their housing programmes houses built of one of the various forms of concrete construction, and, by so doing, not only meet the demand for houses, but also absorb labour that is largely unemployed.

A large proportion of the suggestions submitted to the Committee relate to the application of concrete to house building and, after examination of a great number of proposals, the Committee are of opinion that it is desirable to present at once a second interim report dealing with this form of construction.

General.

The use of concrete for the construction of dwellings in the form of pre-cast blocks is not a new or untried method. There was experience of this material previous to the war, from which the conditions for its safe and successful use were determined. Since the war large numbers of satisfactory concrete dwellings have been erected, in which several methods of construction have been employed. Most of these schemes, however, rely to a lesser or greater degree upon the employment of those expert trades in the building industry which are most scarce and which are essential to the requirements of the present building programme in materials of construction other than concrete. There is a sufficient body of experience to show that comfortable, dry and healthy dwellings, satisfactory in appearance, can be constructed of concrete. Suitable aggregates, representing five-sixths or more of the ingredients required, are to be found in most districts. They require no skilled labour to prepare, and the quantity available is generally plentiful. The capacity for the production of cement is ample. Consequently, there is no likelihood of any shortage in the materials required. The labour needed both for preparation and construction is mainly unskilled, and what skill is necessary for the work can readily be acquired under properly qualified supervision.

In situ Concrete.

One of the best methods which will substantially supplement the output of houses is the use of concrete placed or poured *in situ* between wooden or steel shutters. This method of building walls and partitions is a proved success, but has not been largely employed. The outer layer of a house constructed by this method should be made impervious and the inner layer of porous ingredients, of such a nature as would prevent capillary attraction and the passage of heat. If very dense concrete is used provision must be made for expansion.

Poured concrete of a special character in which the proportion of cement to suitable aggregate is reduced to one in nine is incorporated in housing schemes in Holland. Effective buildings have been constructed by this method up to three stories in height with walls only about 8 in. thick, in which there is a high percentage of voids, and consequent lightness of structure.* In these houses the walls are finished externally by cement rendering or roughcast, and inside by ordinary plaster. These plastered surfaces, together with poured concrete, form the outer wall, and no air space is required. Where hollow walls are desired, a concrete of this character would serve well for the inner layer of a cavity wall, an impervious concrete layer (poured *in situ*) forming the outer surface. The porous material is also very suitable for

[* These houses were described and illustrated in our issue for October, 1923.]

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the partitions which sub-divide the house. In order to minimise plastering as far as possible, in many cases a fair finish can be produced by carefully stopping all holes, rubbing down and applying a cement wash with a brush, and so preparing a surface sufficiently well to take paper or distemper.

Concrete floors, partitions, and roofs have been satisfactorily constructed in some of the existing housing schemes in this country, and linoleum or cork carpet on concrete floors has been in use with hard wear for upwards of twenty years in office blocks. Such floors may be more expensive than wooden ones, but they do not necessitate carpentry work, and the use of steel shuttering will probably avoid the use of plaster work.

A flat roof of concrete does not lend itself so easily to architectural treatment as does the tiled or slated roof. It is possible, however, if it is desired to economise in slating or carpentry work, to make a perfectly good weatherproof concrete roof if covered with a suitable type of bituminous sheeting.

With poured concrete (*in situ*) walls, floors, and roof, the building would have the great virtue of being as vermin-proof as it could be made.

Properly constructed shuttering can easily be fixed, moved, and refixed by other than skilled men, and the concrete can be placed after being either hand or machine mixed by the same class of labour. In this connection there are great possibilities of satisfactory shuttering of steel or other materials being assembled and re-assembled so as to obviate plastering. The question of an efficient system of easily moved shuttering is one of the greatest importance in connection with poured *in situ* concrete houses, but the last word in this connection has not been said, and the offer of some substantial premium for the best and most economical system of shuttering, capable of being erected, though not necessarily constructed, by unskilled labour, should be made.

There should be no prejudice against a concrete building. Concrete is entirely reliable from a strength point of view. It can be mixed and placed either with the ordinary shovel or by mechanical appliances, and the jointing materials are thoroughly well distributed.

To sum up, the Committee is of opinion—

(1) That houses built of poured *in situ* concrete are satisfactory habitations.

(2) That this method of building which economises in bricklaying and plastering enables a building to be completed rapidly and almost entirely by other than skilled labour.

(3) That various improvements have been introduced in the way of giving a pleasing outside surface and a smooth inside surface to such buildings, with a minimum of labour drawn from the skilled building trades, and that further improvements in these directions should be carefully studied and assisted.

(4) That this method of construction promises more houses and more permanent structures than any other alternative to brick construction which it has had before it, and with the least call on the skilled building operatives.

The maintenance costs of such houses will compare favourably with those of any houses.

Slab Construction.

The system of construction which consists of a wall formed of two pre-cast slabs—these being held at the ends by cement concrete pillars or posts either pre-cast or cast *in situ*—has been largely employed and a great many houses of such construction are being incorporated in the various housing schemes now being carried out. They all suffer, however, from the fact that if they are erected by other than skilled labour it is essential that they should be rough-cast or cemented on the outside to make them weatherproof and plastered within to make them pleasing in appearance. On the other hand, if the slabs are so well placed as to make this roughcasting or plastering unnecessary, they require the employment of the skilled bricklayer or mason to set them properly and to make them water-tight and sufficiently smooth on the interior either to carry distemper or to be capable of receiving a paper covering.

All pre-cast slab constructions should have a strong horizontal bond across the cavity, first at the base of the wall and then at the top of the first story and immediately under the roof, composed either of pre-cast blocks overlapping the inner and outer walls, or preferably a continuous course of concrete in which a small amount of reinforcement has been

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placed. Without a bonding course there will probably be vertical cracking at the junction of the slabs and the posts or pillars. The bonding courses should be of such a nature that they will resist on their upper surface the absorption of water, and provision should be made to carry off condensation and prevent capillary passage of moisture from without.

The Committee has seen several examples of a horizontal bond periodically inserted in the height of a wall where such provision has not been made, the bonding courses being either made of tiles or bricks or concrete cast *in situ*. In all such cases trouble on the inner face exists, as indicated by the lack of adhesion of any wall covering on the inner surface. It is a well-known practice in hollow wall brick construction in buildings where a lintel passes right through from the outer to the inner face of the wall to place a lead flashing to prevent damp showing itself through or above the lintel. This trouble can be prevented by making the lintels in two thicknesses over the openings.

A further point requiring attention is that the vertical pillars must be so formed that condensation will not take place inside. Some existing forms are defective on this ground. Great care must also be taken to place the dampcourse at a proper level.

Block Construction.

As regards the enhancement of the output of a bricklayer, and consequent more speedy house construction, it is an acknowledged fact that blocks larger than bricks which are not too heavy, when set by skilled bricklayers or masons, do increase the speed of construction. Therefore, it is clear, within certain limits of weights, that the bigger the unit which the bricklayer is able to handle, the greater will be the equivalent number of bricks laid, and the cheaper the cost.

The Committee urges all interested in house construction to find a light material which should form the mass of a block or slab larger than a brick. Such a block must have an impervious skin, being part of the block, which would make the outer face water resisting. Plain rectangular blocks 3½ in. to 4 in. thick, such as are now being used in various housing schemes, meet the requirements of the above description and make satisfactory work. The point the Committee most desires to emphasise is the necessity for lightness in the unit of the structure, combined with the imperviousness of the exterior face, and the augmentation of the output of skilled bricklayers. If blocks of this kind are set by skilled tradesmen it may be possible to do without plastering inside and rough-casting outside.

Houses formed of pre-cast slabs or pre-cast blocks are quite satisfactory if they are composed of clean and properly mixed ingredients, if a sufficient amount of horizontal bond is provided, and if the vertical pillars (where such are used) are so constructed that condensation on their inner surface cannot take place ; and the use of these methods will enable an important addition to be made to the number of houses at present being produced by the ordinary methods of construction.

Mechanical Plastering.

The Committee directs attention to the mechanical application of a rough-cast exterior to the slab or the block construction by the use of the cement gun, which is being used with satisfactory results in various housing schemes and is largely used in the United States for the exterior coating of wooden houses. As the cement gun can be used so as to give satisfactory results, there seems to be a much greater field for the double slab system assembled by other than craftsmen.

Large Slab Construction.

Another system which is worthy of mention is a poured concrete method of construction which was tried some years ago in Liverpool, and is about to be repeated. It originated with Mr. Brodie, the City Engineer, and involves the erection of buildings out of large concrete pre-cast slabs poured into moulds. The walls are not provided with air spaces, but made of impervious concrete without and breeze concrete on the inner surface. The slabs, weighing several tons, are lifted into place. The method has produced effective houses which have been in use for twenty years, and there seems no reason why such a system should not be adopted in other areas. Comfortable and wholesome homes exist built on this

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system, and the application is one that can be looked to to help the housing problem.

This method of construction would be especially applicable to large apartment dwellings built of steel framework, where large panels could be constructed on the ground and raised from the steel framing of the structure itself to the several floors of a group of tenements. Such panels could be slipped in between the standardised framing formed by the steel structure, and such a method of construction would compare favourably in cost and be more expeditious than if the panels were built of brickwork, which in the past has been the usual method of dealing with steel frame structures.

It is very important that, whether for poured or block concrete, all necessary steps should be taken to ensure that the ingredients to be used, and the manipulation of them, are satisfactory, and the Committee's recommendations are based on the assumption that careful attention must be given to these points.

The present report should not be regarded as exhaustive so far as concrete is concerned. Various systems are still being brought to notice, and, if necessary, the Committee will submit a further interim report.

[We understand that the suggestion of the Committee that a premium be offered for improved types of shuttering has been taken up by the British Portland Cement Association, who are prepared to offer a premium for this purpose.—ED.]

CONCRETE ROADS IN CARDIFF.

DURING a recent visit of the Institution of Municipal and County Engineers to Cardiff, Mr. T. P. Frank, M.Inst.C.E., the City Engineer, gave the following particulars with regard to the methods of laying concrete roads in that city.

The roads recently completed, or in course of construction, are formed, or are being formed, on the two-course principle. The works extend to a length of 1·2/3 miles, and cover 25,000 super yards in seven roadways. The aggregate and sand for the lower course in almost every case is being obtained by washing the old road metal, and by this method appreciable saving is effected in the cost of materials.

The total thickness of the roadway, when reinforced, is usually 7 in., made up of a 5-in. lower course (aggregate from 2 in. downwards) and a 2-in. top course (from ¾-in. chippings downwards), the upper course being laid while the lower coat is still green. The proportions of the lower course are 1 : 2 : 4 and for the upper course 1 : 1½ : 3. The aggregate and sand are graded, and tested at intervals to see that they are free from organic impurities. In the case of roads which are not reinforced the average total thickness of the two courses is 9 in. Where the subsoil is of a clayey nature a 3-in. layer of ashes is rolled in in order to "kill" the clay.

The roads are in most cases formed on the alternate bay type of construction; the joints between the bays are splayed across the roads (i.e., not at right angles with the length of the road), and the bays have, in several instances, been laid of unequal sizes, the larger bays being constructed first. The object of this is to provide for the contraction of a greater mass of concrete before the smaller intermediate bays are constructed. The concrete of both the lower and the upper courses is mixed in a relatively dry condition, and the finished surface, after longitudinal tamping, is screeded transversely with canvas belting.

In the case where the continuous construction has been adopted, expansion jointing material ⅜ in. in thickness is inserted between every 50 ft. to 60 ft. length of roadway. The cross-fall of the surface varies from 1 in 30 to 1 in 50.

The surface of the concrete is protected for the first twenty-four hours from the sun, wind or frost by means of a canvas sheet, and after that period the concrete is covered with a curing coat (which is kept moist) of [illegible] or fine materials, obtained by washing the old material. About four [illegible]n days after the laying of the concrete the surface is treated with 4 to 1 solution of silicate of soda. The solution [illegible] been spread on the roads and brushed in, care being taken that the surface was clean and dry before the application of the solution. This treatment has been repeated on two other occasions at intervals of one day.

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RAPID HARDENING PORTLAND CEMENT.

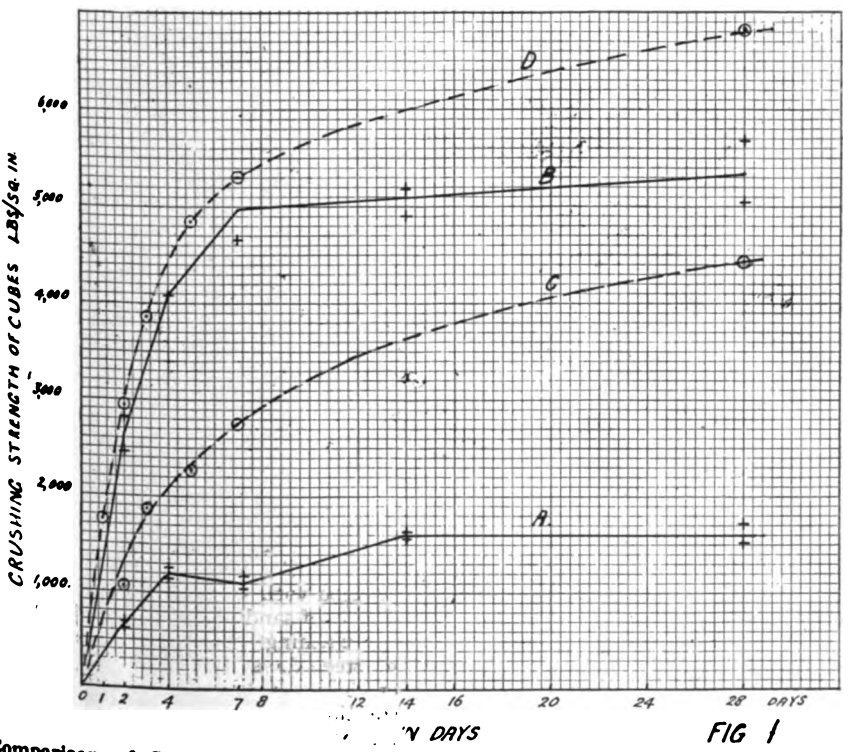
Results of Tests Up to 28 Days.

In our last issue we gave the results of tests up to 14 days on beams and cubes made with the new "Ferrocrete" rapid hardening Portland cement and similar beams and cubes made with ordinary Portland cement, which showed that at less than two days test pieces made under practical conditions with rapid hardening cement attained greater strength than test pieces made under exactly the same conditions with ordinary Portland cement at 28 days.

We have now received a report from Dr. Oscar Faber, O.B.E., D.Sc., M.Inst. C.E., who designed the test beams, giving the results of the tests up to 28 days, as shown on the accompanying curves, and some comments on the tests. In the course of this report Dr. Faber says:

It was decided that these tests should not only be laboratory tests, which are of course open to some suspicion by practical users of cement on the grounds that laboratory tests do not represent practical conditions, and therefore in addition to the usual tensile and crushing tests of the cement and cement mortar and of concrete cubes made in the laboratory we designed a series of test beams of reinforced concrete 16 ft. long, which were all reinforced identically so that the only difference between them was that half were made with ordinary cement and half were made with the special quick hardening cement.

These were made by practical workmen



Comparison of Strength of Rapid Hardening and Ordinary Portland Cement Concrete.

A = Average Portland Cement Concrete made by Contractors' Workmen.
B = "Ferrocrete" (Rapid Hardening) Portland Cement Concrete made by Contractors' Workmen.
C = Average Portland Cement Concrete made in Laboratory.
D = "Ferrocrete" (Rapid Hardening) Portland Cement Concrete made in Laboratory.

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in the establishment of Messrs. Jackaman & Sons, Ltd., of Slough, under practical conditions. Cubes of the actual concrete used in these test beams were also made up, and both the test beams and the cubes were then sent to a laboratory for testing purposes only, as undoubtedly the actual testing work is more accurately done in the laboratory and does not in any way detract from the practical conditions under which these test specimens were made. A drawing illustrating these test beams, showing the reinforcement, the size of the beam, and the way in which the load was applied, was given in our last issue. Preliminary calculations indicated that it would be of no use making these test beams with the ordinary small percentage of steel used in practice for reasons of economy, because the yield point of the steel would then be reached long before the compressive strength of the concrete, with the inevitable consequence that all the beams would yield and fail at a load depending solely on the properties of the steel, and would therefore give no information about the strength of the concrete. For example, supposing that the usual 0·675 per cent. of steel were used, which corresponds to a working stress of 16,000 lbs. per sq. in. in the steel and 600 lbs. per sq. in. in the concrete. If the yield point of the steel is 40,000 lbs. per sq. in., which is a common figure, then clearly the beams would fail when the stress in the concrete was only 600 lbs. per sq. in., increased in the ratio of 40,000 lbs. to 16,000 lbs., or in other words, 1,500 lbs. per sq. in., and no advantage would be indicated by any test beams having concrete of greater strength than this because the failure of the steel would take place first.

When it is mentioned that with the new cement a crushing strength exceeding 1,500 lbs. per sq. in. is obtained even at the age of two days, and reaches figures many times this value at a greater age, it will be obvious that such test specimens will be quite unsuitable for a true comparison of these concretes. This explains why a considerably greater percentage has been adopted in the test specimens so that failure shall in every case occur by

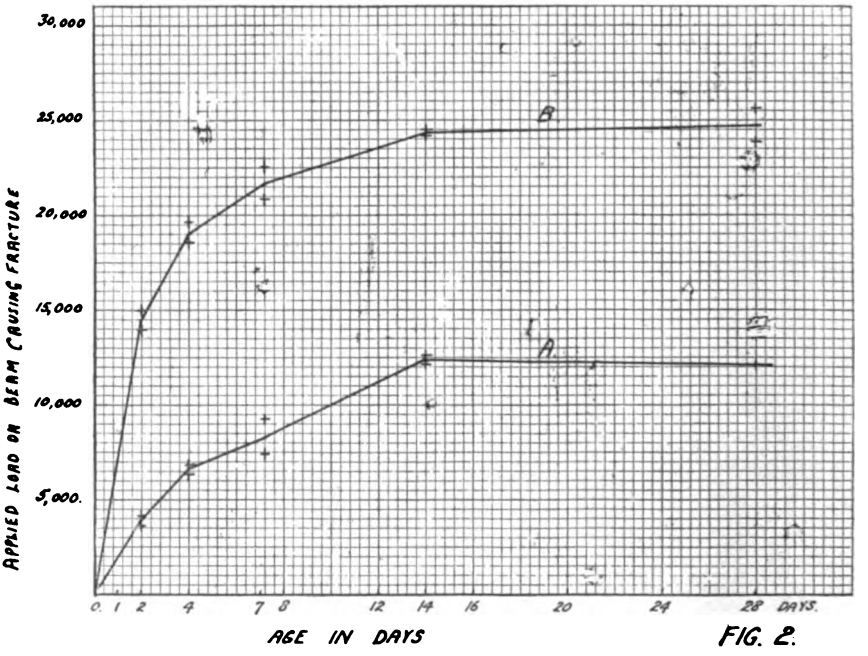

FIG. 2.

Comparison of Strength of Rapid Hardening and Ordinary Portland Cement Concrete.

A = Average Portland Cement Concrete.
B = "Ferrocrete" (Rapid Hardening) Portland Cement Concrete.

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crushing of the concrete earlier than by the yielding of the steel, and as the same reinforcement is used consistently throughout in all the beams the results are clearly strictly comparable.

Altogether 24 beams were made, 12 with ordinary cement and 12 with "Ferrocrete" rapid hardening Portland cement. To ensure identical weathering all the beams were made on the same day under identical conditions and stored under identical conditions. Ham River ballast and sand were used in the proportions of about 5½ to 1, but specified more accurately, and the water was carefully controlled. Mixing was done by hand and ½ in. of cement mortar placed at the bottom of the mould before the concrete proper was placed. This is in accordance with the best practice in reinforced concrete construction. The steel was ordinary commercial mild steel to British Standard Specification, and to ensure comparative results all the rods were from the same rolling. The steel had a yield point of 21·3 tons per sq. in. and a breaking strength of 31·5 tons per sq. in. in the case of the 1¼-in. rods. The ⅜-in. rods had a yield point of 24·1 tons per sq. in. and a breaking strength of 31·9 tons. The beams were designed so as not to fail in shear.

Test cubes were made from each batch of concrete used in the beams and numbered similarly, stored under identical conditions, and tested at the same time as the beams. When the beams were due to be tested they were carted to the laboratory at the City and Guilds (Engineering) College, South Kensington, where the excellent testing facilities and the great advantage of the experience in testing of Professor S. M. Dixon and Professor J. Pursor had been secured. To prevent any risk of the upper flanges of the beams cracking during transit, this flange not being reinforced, temporary bolts ¾ in. diameter with washer plates at the two ends of the beams were used to put the top flange into a state of compression just sufficient to overcome any accidental tension stresses which might occur in transit. When the beams arrived at the laboratory and had been put into the

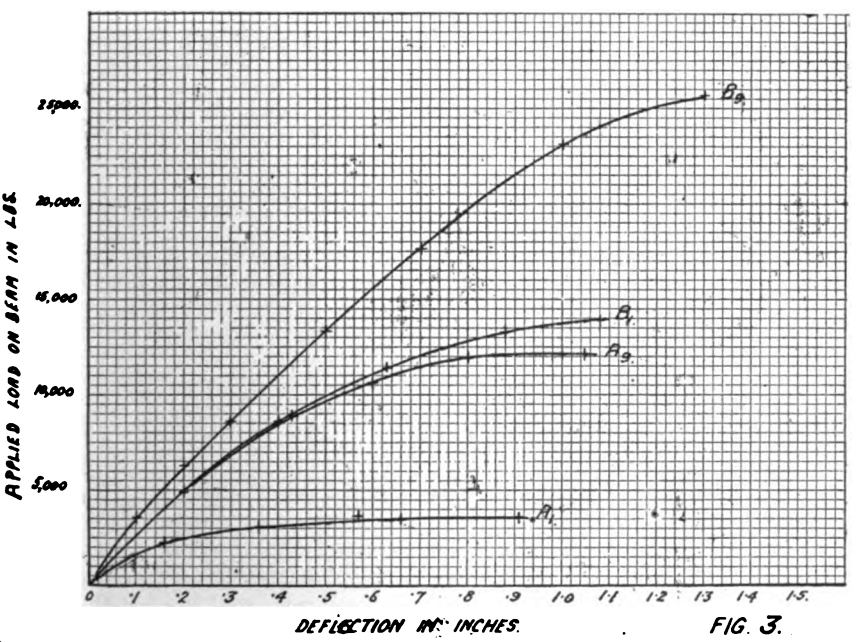

FIG. 3.

Comparison of Strength of Rapid Hardening and Ordinary Portland Cement.

A_1 = Beams of Ordinary Concrete 2 days old.
A_9 = Beams of Ordinary Concrete 28 days old.
B_1 = Beams of "Ferrocrete" (Rapid Hardening) Portland Cement 2 days old.
B_9 = Beams of "Ferrocrete" (Rapid Hardening) Portland Cement 28 days old.

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testing machine these bolts were removed and returned, to be used again on the next beams.

Results of Tests.

The results of the tests are represented in Figs. 1, 2 and 3.

Fig. 1 is a curve in which the crushing strength of the cubes is plotted against the age in days. Curve A, being the strength for the ordinary concrete cubes prepared by the contractor, gives a strength in 4 days of about 1,200 lbs. per sq. in., increasing in 28 days to about 1,600 lbs. per sq. in. There are the variations which are commonly found in curves of this kind, and the results are, of course, lower than the corresponding laboratory tests for the same materials and the same cement. This is only what we should expect.

As a comparison to Curve A, Curve B gives the results of the concrete made with "Ferrocrete" rapid hardening Portland cement, also by the contractor's workmen. It will be seen that at 4 days the results are approximately 4,100 lbs. per sq. in., increasing to about 5,350 lbs. at 28 days. In other words, at 4 days the concrete is over 3½ times as strong, and at 28 days it is over 3¼ times as strong. On the same figure are given two other curves, C being the test cubes on ordinary cement from the same works prepared by Messrs. Riley, Harbord & Law, made under laboratory conditions and kept at standard temperatures for the whole length of their maturing. Curve D shows the strength of similar cubes made by the same firm of testers, but of the same rapid hardening Portland cement as was used for the test beams. It will be seen that Messrs. Riley, Harbord & Law's tests, which are shown dotted on the figure, are in both cases higher than those made by the practical workmen of the contractor, and this is only to be expected. The difference is particularly marked in the period between 14 and 28 days and is largely due to the fact that the test beams and the corresponding cubes made by the contractor were taken out into the cold December weather when they were 14 days old, whereas the cubes made by the testing firm were kept under standard temperature conditions to the end. There is no doubt that had the test specimens been kept in a moist damp temperature for the whole period they would have shown a much bigger increase in strength, and the Curves A and B on *Fig.* 1 would have approximated more closely to Curves C and D respectively.

The curves in *Fig.* 2 would also have shown a greater rate of growth, as it is well known that concrete grows very little in strength at the low temperatures in cold winter weather. *Fig.* 2 gives the corresponding results, but those of the test beams instead of the cubes. Curve A is the load causing fracture on the beams made with ordinary concrete, and Curve B the breaking load on the beams made with rapid hardening cement. It will be seen that at 4 days the ordinary concrete beams carried about 6,500 lbs. and at 28 days about 12,200 lbs. The corresponding figures for the rapid hardening concrete are 19,000 lbs. at 4 days and 24,750 at 28 days. The beams made of the rapid hardening cement are therefore approximately 3 times as strong as the other at 4 days and approximately twice as strong at 28 days.

It is to be specially noted that the curves do not indicate any reduction in strength, even relatively, and rather indicate that this ratio of strength of twice would be maintained at greater ages.

Fig. 3 is a curve showing the relationship between the applied load and the deflection of the beam in inches. To avoid confusion only four curves are shown on this diagram, A1 and A9 being beams of ordinary concrete tested at the age of 2 days and 28 days respectively, and B1 and B9 being the results of test beams made with the rapid hardening cement tested at the age of 2 days and 28 days respectively. It will be seen that in all cases a deflection of somewhere just over 1 in. is reached at fracture, independently, or almost so, of whether the beams carried a load of about 3,500 lbs., as in the case of A1, or 25,600 as in the case of B9. In other words, the new cement not only carries a far greater stress than the ordinary concrete, but carries this increased load at practically no increase of deflection. This means that its modulus of elasticity is increased roughly in proportion to its increased strength; to the practical man it means that a beam made with "Ferrocrete" rapid hardening Portland cement will carry two or three times the load of a beam made with the ordinary cement without an increase in deflection, which for some purposes is quite an important considera-

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tion. The result is an especially interesting one and bears on the regulation in the London County Council regulations for reinforced concrete under the 1909 Act, in which the modulus of elasticity is made to depend on the crushing strength of the concrete, a theory which is strikingly borne out by these tests, whether that increase of strength is obtained by the increasing age of the specimen or whether it is obtained by the use of the superior cement.

General Conclusion.

The results indicate that the new cement is a product of very great practical importance indeed, and it will not only have special importance in cases where an early strength is required (for example, in the case of road construction, special surface finishes requiring to be subjected to wear at an early date, and places where early removal of centering and temporary supports are required, as well as where ferro-concrete piles are required to be driven at an early date), but the greatly increased strength of the material even at later dates should justify substantially greater working stresses with this new material and so make it specially useful and economical for reinforced concrete columns, beams, and structures generally. It will of course be important to get local authorities to make provision for higher working stresses for this stronger material before full advantage of its valuable properties can be taken in many structures subject to such regulations, and this is a matter which ought to have the prompt attention of scientific bodies.

QUESTIONS AND ANSWERS RELATING TO CONCRETE.

REINFORCED CONCRETE FLOOR.

QUESTION.—I wish to lay a reinforced concrete floor 10 ft. by 10 ft. over a yard and leading from the first floor of a house to the first floor of a shop, as per enclosed blue-print [not reproduced]. Will you please advise me as to the depth of the concrete slab, reinforcement necessary, etc.—J. D. L.

ANSWER.—The particulars given are not very explicit and various assumptions have to be made in order to make the necessary calculations for the reinforced concrete floor. The letter states that the floor is 10 ft. by 10 ft., whereas the blue print gives figured dimensions of 9 ft. 3 in. and 6 ft. 9 in. The dimensions on the plan have been taken as correct. Again it is not clear from the particulars given whether a bearing for the slab is available on the adjoining wall, which is apparently the main wall of the building, and it is assumed that the outer wall of the bathroom is a new wall 9 in. thick that must be carried by a reinforced beam, but no mention is made of this and the drawing is not clear on this point.

Assuming that the outer wall is carried by a new reinforced concrete beam, and that the slab can take a bearing on this and on the main wall above referred to, then the floor can be constructed with a reinforced concrete slab having a total thickness of 4 in. and reinforced with ⅜-in. diameter rods spaced at 6-in. centres. These rods would run across the 6 ft. 9 in. width and be hooked at the ends. At each end it would be advisable to crank up every alternate rod at a point 16 in. from the support to bring it into the upper surface between that point and the support. Distribution rods should be provided at right angles to the ⅜-in. diameter rods, these being ¼-in. diameter rods spaced at about 18-in. centres and running across the 9 ft. 3 in. width.

The beam under the wall will require a total depth of 15½ ins. and a width of 9 in., with two ¾-in. diameter rods as reinforcement on the lower surface. The top of this beam should be flush with the top of the floor slab and placed central under the wall. Stirrups or anchors of small round or flat steel should be provided at 12-in. centres throughout the length of the beam, these being placed under and around the main rods and extended up to within 1½ in. from the top surface of the beam.

A bearing of at least 9 in. should be provided at the ends of the beam, and it would appear from the drawing that there would be no difficulty in this. The concrete throughout must be mixed in the proportions of 1 part good Portland cement, 2 parts clean sharp sand, and 4 parts clean hard coarse aggregate, and the work must be executed in accordance with good practice.

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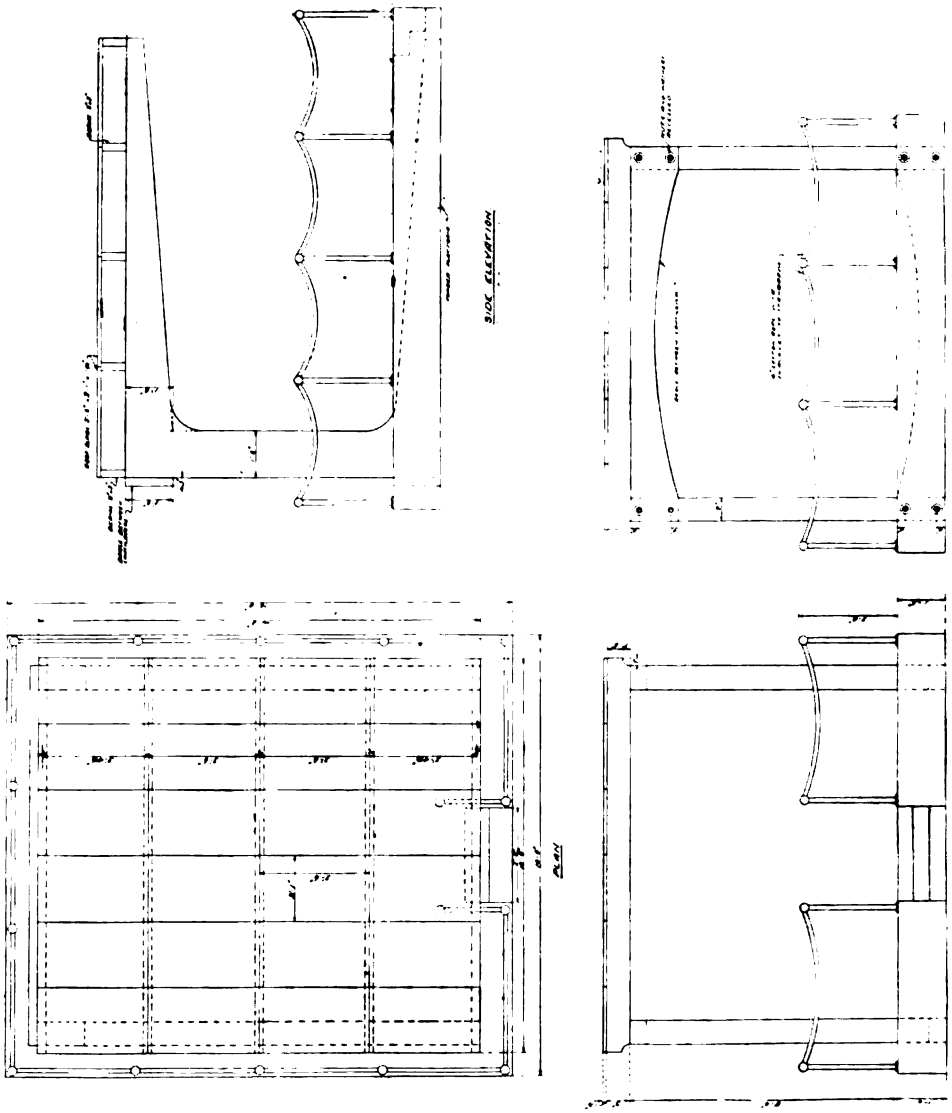

TESTS ON ALUMINOUS CEMENT: FIG. 1. (See p. 91.)

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TESTS ON ALUMINOUS CEMENT.

By Oscar Faber, O.B.E., D.Sc., M.Inst.C.E.

In the early part of last year the writer was asked by the Lafarge Aluminous Cement Co., Ltd., to design an exhibit for their stand at the Building Trades Exhibition at Olympia. This exhibit took the form shown in *Fig.* 1, from which it will be seen that it consisted of two bent beams or large cantilevers, supporting five rectangular beams spanning across from one to the other, and carrying slabs 1½ in. thick, forming the roof of the stand. Lateral swaying of the construction was prevented by two arch-form braces bolted on at the end. The sizes and details of reinforcement are given in *Fig.* 2.

The concrete was gauged as follows: ¾-in. ballast, 7 cu. ft.; sand, 4 cu. ft.; Ciment Fondu, 200 lbs. A smaller quantity of cement would have given all the strength required, but in the writer's view it is not advisable on reinforced work with such small members to use a leaner mix, not only because of the risk of corrosion of the reinforcement if the concrete fails to be waterproof, but also because a good surface finish was desired. This was achieved to a high degree of perfection, as is indicated in the photographs, and no touching up of any kind was resorted to once the framework had been struck.

The remarkable part of the performance lay not only in the somewhat bold proportions of the members, but also in that the members were freely handled and erected only two days old, which with ordinary cement would not have been advisable in less than two weeks at the earliest, since the members were fully stressed to their safe working stresses under their dead load only.

The results of crushing tests of cubes made at the same time as the concrete members of the same concrete were as follows:—

	lbs. per sq. in.
48 hours	5,330
72 ,,	6,030
7 days	8,240
90 ,,	9,230

It will be seen that even at two days the results were greatly superior to what could be achieved with ordinary cement, used in the same proportions, at the age of twenty-eight days, and it is of course this feature wherein the chief interest of this material lies.

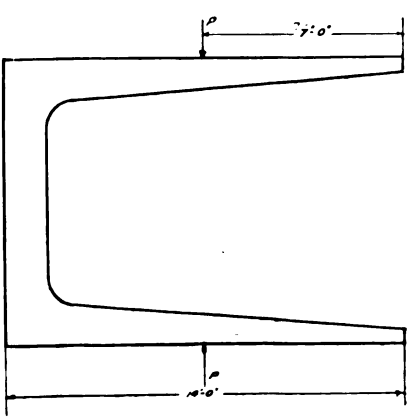
At the conclusion of the Exhibition, Professor S. M. Dixon, of the City and Guilds (Engineering) College, South Kensington, very kindly offered to test the specimens, and they were accordingly sent to him for this purpose and tested in his laboratory. The results have since been analysed by the writer and are given in what follows. The records of the experiments are made more valuable and interesting by the excellent photographs kindly taken by Professor Purser and reproduced here.

Test No. 1, Cantilever No. 1.

The two arms of the cantilever were pulled together by means of tie-bars placed half-way along the span connected to a hydraulic jack (under the floor) with a 5-in. diameter ram and a pump worked by hand from the floor of the laboratory and provided with a hydraulic pressure gauge (*see Fig.* 3). Deflections were measured by a micrometer at the end of the cantilevers observed through a magnifying telescope. The loads and deflections are given in Table 1.

TABLE 1.

NOTE.—Load applied 7 ft. from end by link drawing the two free ends together. Deflections noted are the mutual deflections or approachments of the ends.



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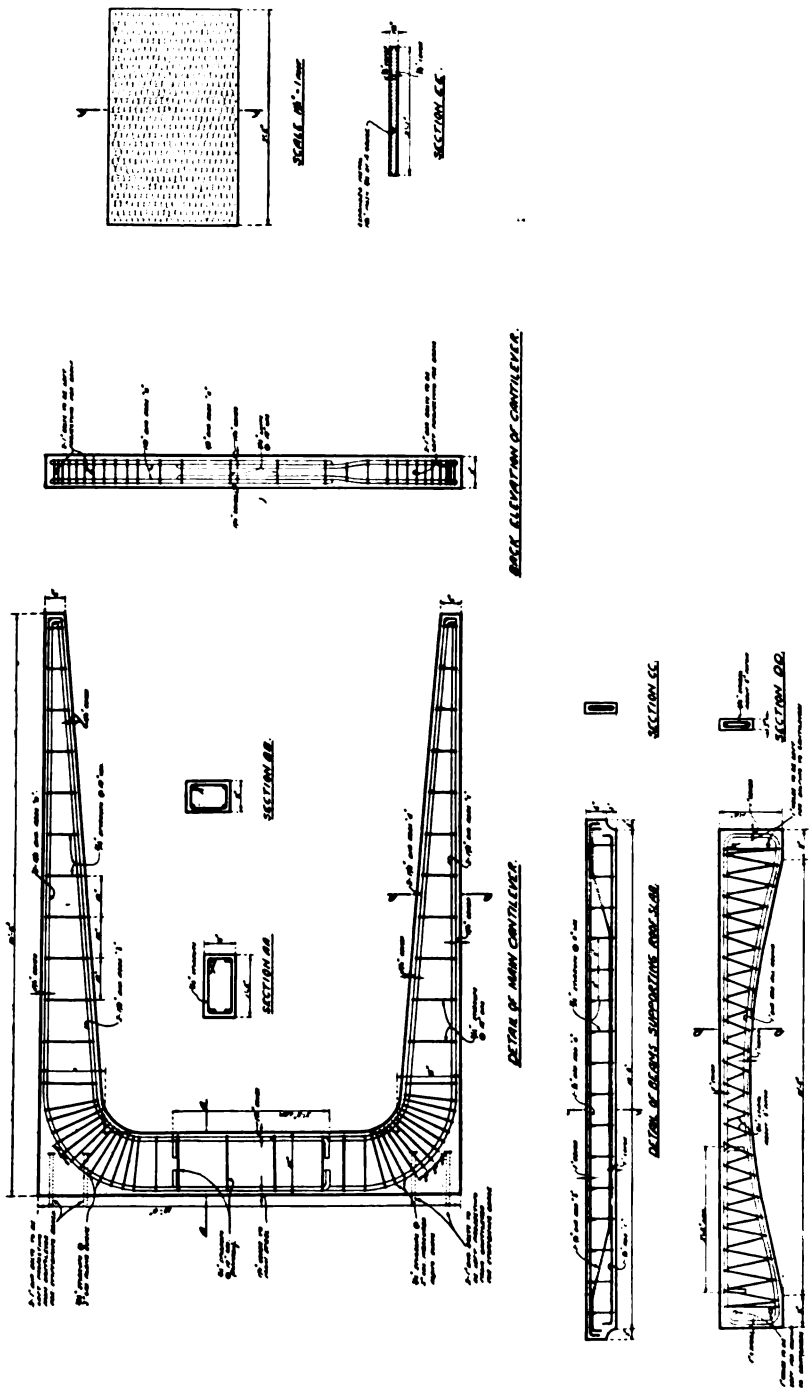

TESTS ON ALUMINOUS CEMENT: Fig. 2.

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Load P lbs.	Deflection inches.	Load P lbs.	Deflection inches.
392	0·01	7,850	1·04
785	0·04	8,240	1·13
1,177	0·07	8,630	1·21
1,570	0·09	9,025	—
1,964	0·13	9,420	1·37
load released.	perm deflection.	9,810	1·45
0	0·03	release	0·37 (permanent)
785	0·05	1,964	0·51
1,570	0·11	3,924	0·75
1,964	0·13	5,886	0·99
2,354	0·17	7,850	1·23
2,748	0·19	9,810	1·49
3,140	0·23	10,200	1·57
3,530	0·29	10,590	1·63
3,924	0·33	10,980	1·71
4,315	0·41	11,380	1·81
4,708	0·45	11,770	1·89
5,100	0·53	12,160	2·01
5,493	0·59	12,550	2·11
5,886	0·67	12,950	2·23
6,280	0·75	13,340	2·35
6,650	0·83	13,740	2·47
7,000	0·91	release	0·68
7,455	0·99		

15,710 maximum load.
Load at photograph, 11,770 lbs.

Test No. 2, Cantilever No. 2.

Exactly similar tests were carried out with Cantilever No. 2 with results as given in Table 2. The deflections were not measured in this case, but the beam behaved almost exactly as No. 1.

TABLE 2.

Load P in lbs.	Load P in lbs.	Load P in lbs.	Load P in lbs.
392	2,748	7,060	5,886
785	3,140	7,455	7,850
1,177	3,530	7,850	9,810
1,570	3,924	8,240	10,200
1,964	4,315	8,630	10,590
release	4,708	9,025	10,980
785	5,100	9,420	(cracking)
1,570	5,493	9,810	11,380
1,964	5,886	release	11,770
2,354	6,280	1,964	12,160
	6,650 (crack)	3,924	

Maximum load, 14,970 lbs.

Analysis of Results of Nos. 1 and 2.

	lbs.
Maximum load No. 1 . .	15,710
,, ,, No. 2 . .	14,970
Sum	30,680
Mean	15,340

From *Fig.* 2 it will be seen that the vertical part of the member, which is where failure occurred, is reinforced with two $1\frac{1}{8}$-in. diameter rods on each side, and $14\frac{3}{8}$ in. apart in a member 18 in. by 9 in. in section. The forces were then as in *Fig.* 4.

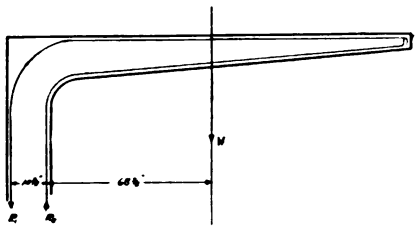

FIG. 4.—DIAGRAM OF FORCES.

Safe tension in R_1 (limiting the steel stress to 16,000 lbs. per sq. in.) is

R_1 = 2 sq. in. × 16,000 lbs. per sq. in.
= 32,000 lbs.

whence

$$\text{Safe load } W = 32{,}000 \times \frac{14\frac{3}{8}}{68\frac{3}{8}} = 6{,}700 \text{ lbs.}$$

Breaking load W applied = 15,340 lbs.
dead weight of cantilever

$$14 \text{ ft.} \times \frac{12 \text{ in.} \times 9 \text{ in.}}{144} \times 150 = 1{,}570 \text{ lbs.}$$

16,910 ,,

$$\text{Therefore factor of safety} = \frac{\text{breaking load}}{\text{safe load}}$$

$$= \frac{16{,}910}{6{,}700} = 2{\cdot}53$$

In other words, the stress in the steel when failure occurred was

16,000 × 2·53 = 40,500 lbs. per sq. in.

It is interesting to note that the elastic limit of the steel was found by test to be 20·55 tons per sq. in., or 46,000 lbs. per sq. in.

Report on Tensile Test of Steel Reinforcement.

	From cantilever.	From cantilever.	From beam.	From beam.
No. . .	1	2	3	4
Marks . .	..	..	∴	None
Length between gauge points .	8 in.	8 in.	3 in.	3 in.
Diameter .	0·850	0·875	0·373	0·370
Area . .	0·567	0·601	0·1093	0·1075

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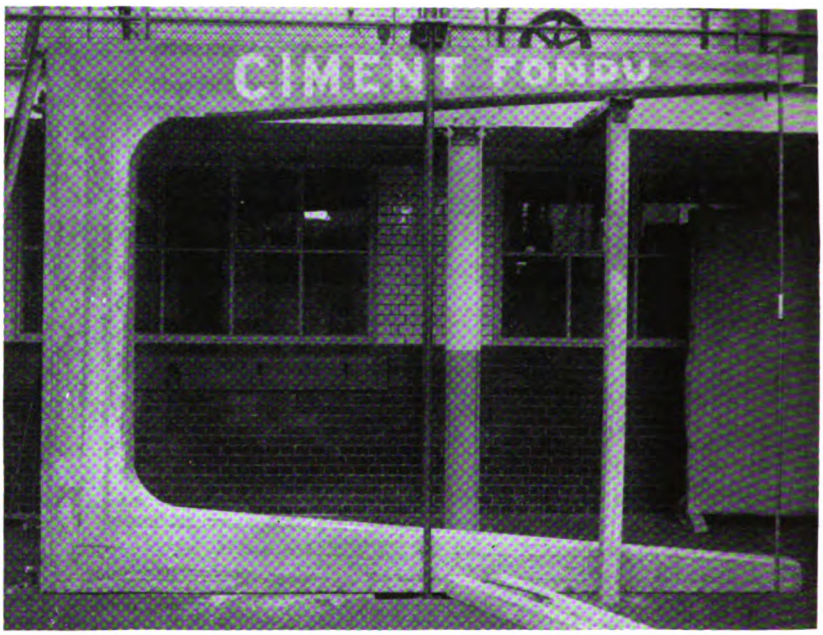

FIG. 3.—BEFORE TEST.

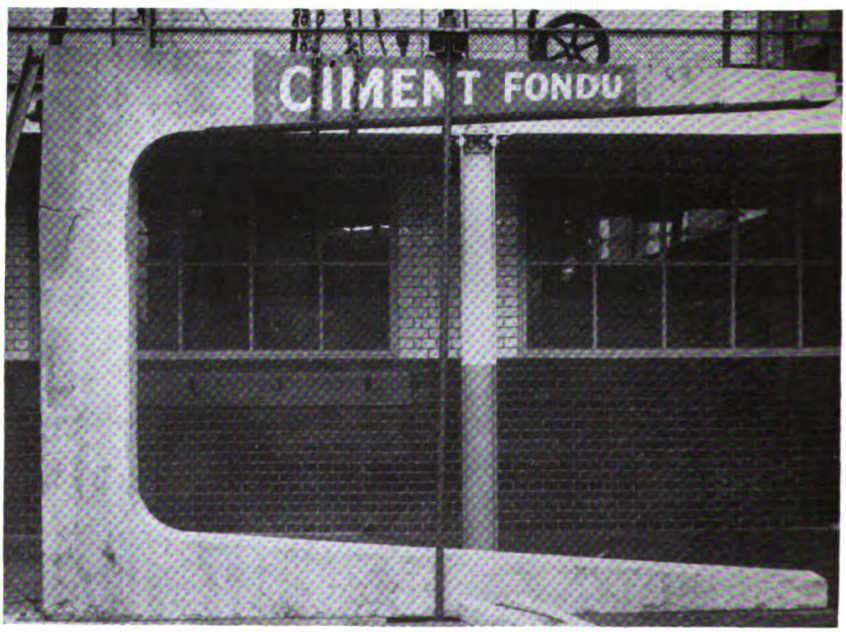

FIG. 5.—AFTER TEST.

CANTILEVERS UNDER TEST.

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Before Test.

	From cantilever.	From cantilever.	From beam.	From beam.
No. . .	1	2	3	4
Length between gauge points .	10·1	10·32	3·87	3·68
Diameter at rupture	0·50	0·50	0·22	0·22
Area at rupture	0·196	0·196	0·038	0·038
After Test.				
Elastic limit, tons per sq. in. .	20·7	20·4	28·1	23·7
Ultimate strength, tons per sq. in. .	27·1	26·9	32·3	32·2
Extension in 8 in. per cent.	26·25	29·0	29·0	22·67
Reduction of area per cent.	65·5	67·4	65·0	64·7

The actual failure appeared to be due to the slipping of the rods where they lapped in the vertical member.

The superficial area of the two rods $1\frac{1}{8}$ in. diameter for a length of 3 ft. 9 in. is $A = 2 \times \pi \times 1\frac{1}{8}$ in. $\times$ 45 in. $= 32{\cdot}2$ sq.in.

The breaking force R_1 was

$$16{,}910 \times \frac{68\frac{3}{8}}{14\frac{3}{8}} = 81{,}000 \text{ lbs. per sq. in.}$$

Hence, neglecting any strength in the hooks at the ends of the lap, the adhesion stress at failure was

$$f = \frac{81{,}000}{32{\cdot}2} = 250 \text{ lbs. per sq. in.}$$

The adhesion stress at working or safe loads (when the tensile stress was 16,000 lbs. per sq. in.) was 100 lbs. per sq. in., so that the factor of safety on adhesion was

$$\frac{250}{100} = 2{\cdot}5$$

There is no object in calculating compressive or shear stresses, as failure clearly occurred on the tension side.

Fig. 5 shows a cantilever after failure, and the cracks at the hooked ends of the lapped rods are plainly visible.

The tests were extremely satisfactory in the following respects :—

(1) The close concordance between the results of the two cantilevers (15,710 and 14,970 lbs. respectively).

(2) The extraordinary way the beam continued to carry its maximum load long after it had cracked and when the deflection was great. This of course makes for reliability and plenty of warning before failure occurs.

(3) The factor of safety being exactly what was expected. (The writer had previous to the tests advised the Company that failure would occur on the tension side and that the factor of safety would be about 2·5.) The importance of this lies in the fact that our ordinary theories when intelligently applied are applicable to the new material and can be used with safety in experienced hands.

Test No. 3.—Horizontal part of Cantilevers tested as a Beam.

The cantilevers were now cut in half through the vertical portion, and the horizontal portions were supported at the ends and tested by the application of a central point load. This gave four specimens. The span was 12 ft., and spherical bearings were used at the ends. The arrangement is shown in *Figs.* 6 and 7. The results are given in Tables 3 to 6.

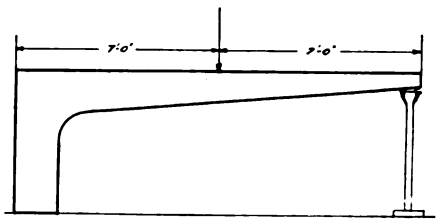

TABLE 3.

Test of top of Cantilever No. 1.

Load lbs.	Deflection inches.	Load lbs.	Deflection inches.
370	0	6,370	0·165
870	0·023	6,870	0·177
1,470	0·032	7,370	0·190
1,840	0·041	7,870	0·204
2,430	0·061	8,350	0·215
2,850	0·070	8,920	0·229 (Crack 6 ft. from small end)
3,460	0·084		
3,930	0·099		
4,300	0·110	9,470	0·245 (Cracks 5 ft. 10½ in. and 6 ft. 2 in. from small end)
4,870	0·124 (seems to be yielding slightly)	9,970	0·264
5,270	0·130	10,870	0·285
5,870	0·152		

Maximum load, 22,150 lbs.

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FIG. 6.

FIG. 7.
HORIZONTAL PART TESTED AS BEAM.

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TABLE 4.

Bottom of Cantilever No. 1.

Load lbs.	Deflection inches.	Load lbs.	Deflection inches.
1,450	0·02	10,450	0·21
3,450	0·05	11,450	0·23
5,450	0·10 (audible cracks)	13,450	0·27
7,450	0·14 (cracks underside)	15,450	0·32
8,350	—	17,450	0·37
9,450	0·18	18,450	0·39
		21,450	0·54

Maximum load, 24,990 lbs.

TABLE 5.

Top of Second Cantilever.

Load lbs.	Deflection inches.	Load lbs.	Deflection inches.
1,330	0·01	13,330	0·24
3,330	0·04	15,330	0·28
5,330	0·07	17,330	0·32
7,330	0·12 cracks (first)	19,330	0·37
9,330	0·16	21,330	0·42
11,330	0·20	23,930 (*failed*)	

TABLE 6.

Bottom of Second Cantilever.

Load lbs.	Deflection inches.	Load lbs.	Deflection inches.
1,360	0·02	13,360	0·23
3,360	0·05	15,360	0·27
5,360	0·05	17,360	0·31
7,360	0·12 (first crack)	19,360	0·35
9,360	0·15	21,360	0·41
11,360	0·19 (centre crack)	22,940 (*failed*)	

Analysis of Test No. 3.

The maximum loads from the four tests are as follows :—

	lbs.
	22,150
	24,990
	23,930
	22,940
Sum	94,010
Mean	23,502

Weight of beam between supports

$$12 \text{ ft.} \times \frac{12 \times 9}{144} \times 150 = 1{,}350 \text{ lbs.}$$

Equivalent central point load = 675 lbs.
Total central breaking load = 24,177 lbs.

The section of the beam at midspan was 13 in., and was reinforced with two $1\frac{1}{8}$-in. diameter rods $9\frac{3}{8}$ in. apart.

The bending moment at fracture was

$$M = \frac{Wl}{4} = \frac{24{,}177 \times 144}{4} = 872{,}000 \text{ in.-lbs.}$$

Whence the stress in the tension rods (area 2 sq. in.) was

$$t = \frac{872{,}000}{9\frac{3}{8} \times 2 \text{ in.}^2} = 46{,}600 \text{ lbs. per sq. in.}$$

This compares with the measured elastic limit of 46,000 lbs. per sq. in.

As compared with a safe working stress of 16,000 lbs. per sq. in., the factor of safety was

$$\frac{46{,}600}{16{,}000} = 2{\cdot}91.$$

No calculations of shear, adhesion, or compression are of much value, as failure clearly occurred by tension. It is perhaps worth noting that the shear at the support where the beam is shallowest was

$$11{,}751 + 675 = 12{,}426 \text{ lbs.}$$

corresponding to

$$\frac{12{,}426}{4\frac{3}{4} \text{ in.} \times 9 \text{ in.}} = 290 \text{ lbs. per sq. in.}$$

(*Note.*—$4\frac{3}{4}$ in. is the radius arm of the beam over the support at the shallow end.)

This is rather more than a beam of ordinary concrete unreinforced in shear would resist, and is accounted for by the increased tensile strength of the material.

Tests No. 4.

Five rectangular beams, 9 in. by 3 in. by 12 ft. 6 in. long, tested under central point load on a 9-ft. span. All deflections measured on 7·5-ft. span. The results are given in Tables 7 to 11.

TABLE 7.

Beam No. 1.

1st crack . . . 3,650 lbs. centre.
Final load . . . 4,710 ,, ,,
4,200 lbs. carried till deflection at centre = 5 in.

TABLE 8.

Beam No. 2. Weight 300 lbs.

Load lbs.	Deflection inches.	Load lbs.	Deflection inches.
500	0·03	2,500	0·18
1,000	0·06 (centre crack)	3,000	0·22
1,500	0·10	3,500	0·27
2,000	0·14	4,000	0·32
		4,360	(crack)

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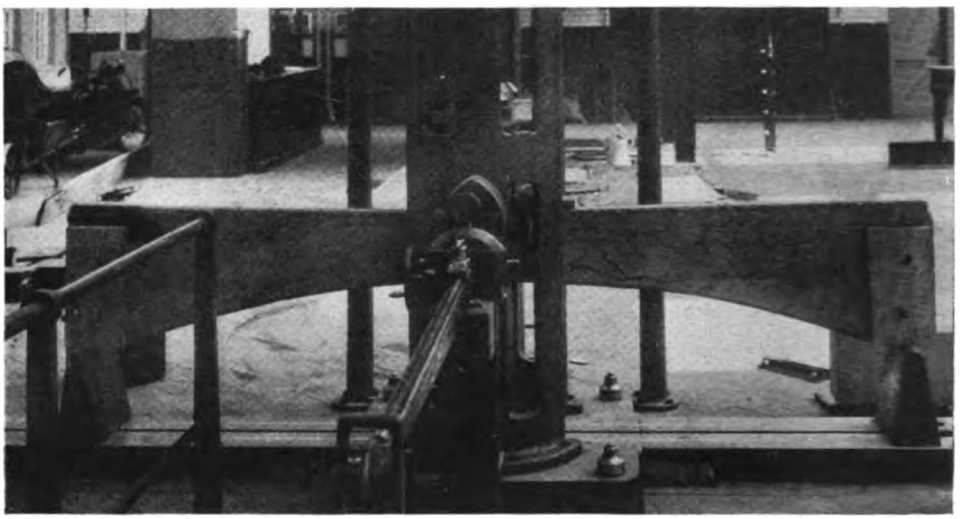

FIG. 8.

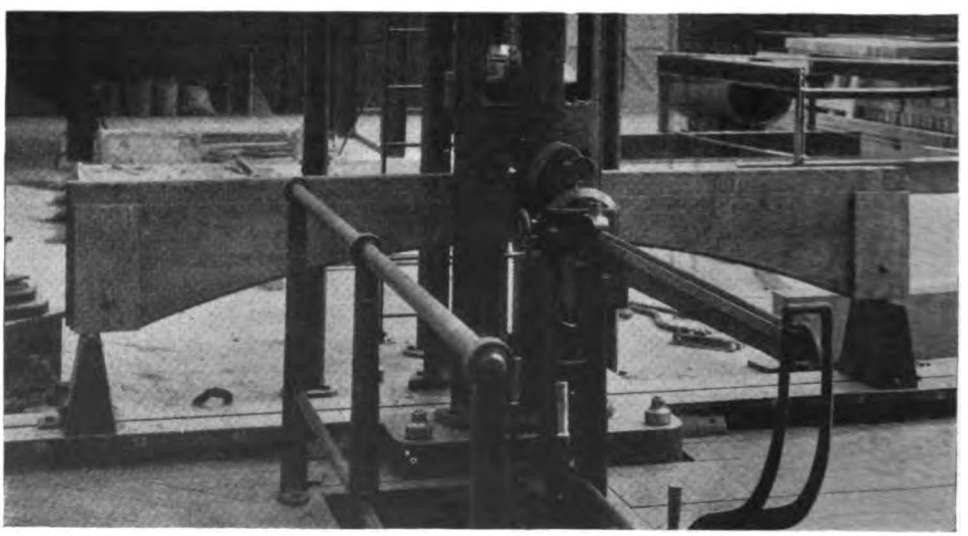

FIG. 9.

TESTS OF ARCHED BEAMS.

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TABLE 9.

Beam No. 3. Weight 310 lbs.

Load lbs.	Deflection inches.	Load lbs.	Deflection inches.
490	0·02	2,490	0·17
990	0·02	2,990	0·22
	(crack at centre)	3,490	0·24
1,490	0·10	3,990	0·31
1,990	0·13	4,360 (*failed*)	
	(crack at centre)		

TABLE 10.

Beam No. 4. Weight 330 lbs.

Load lbs.	Deflection inches.	Load lbs.	Deflection inches.
470	0·03	2,470	0·17
970	0·05	2,970	0·20
	(crack)	3,470	0·23
1,470	0·08	3,970	0·27
1,970	0·13	4,370 (*failed*)	

TABLE 11.

Beam No. 5. Weight 310 lbs.

Load lbs.	Deflection inches.	Load lbs.	Deflection inches.
490	0·03	2,990	0·18
990	0·06	3,490	0·22
	(crack at centre)	3,990	0·27
1,490	0·08	4,490	0·29
1,990	0·12	4,750 (*failed*)	
2,490	0·15		

The remarkable cohesion of these beams is illustrated by the fact that Beam No. 1, which failed at 4,710 lbs., continued to carry 4,200 lbs. when the deflection had been increased to 5 in. A beam of ordinary concrete would not have given this result, as the very high stresses on the compressed edge which result after the yielding of the steel on the tension side (and the resulting raising of the neutral axis up to near the compressed edge) would have produced complete failure much earlier.

Analysis of Tests No. 4.

The results of the five beams are as follows :—

Applied breaking loads.

	lbs.
	4,710
	4,360
	4,360
	4,370
	4,750
Sum	22,550
Mean	4,510

Weight of beam between supports

$$9 \text{ ft.} \times \frac{27}{144} \times 150 = 250 \text{ lbs.}$$

Equivalent load at midspan = 125 lbs.
Total breaking load = 4,635 lbs.
Area of steel in tension = 0·4 sq. in.
depth of C.G. of steel = $7\frac{1}{2}$ in.
Radius arm = $7\frac{1}{2}$ in. × 0·88 = 6·6 in.
Bending moment at failure

$$= \frac{Wl}{4} = \frac{4{,}635 \times 108 \text{ in.}}{4} = 125{,}500 \text{ in.-lbs.}$$

Stress in tension at failure

$$t = \frac{125{,}500}{0{\cdot}4 \times 6{\cdot}6} = 47{,}600 \text{ lbs. per sq. in.}$$

The observed elastic limits were

	28·1	tons per sq. in.
	23·7	,, ,, ,,
Sum	51·8	,, ,, ,,
Mean	25·9	,, ,, ,,
	=58,000	lbs. per sq. in.

The value of 28·1 is unlikely in a mild steel specimen, and open to doubt. The other value gives

$$23{\cdot}7 \ t/\text{in.}^2 = 53{,}000 \text{ lbs. per sq. in.}$$

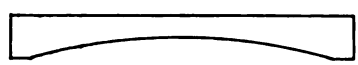
The factor of safety (as compared with a safe working stress of 16,000 lbs. per sq. in.) is

$$\frac{47{,}600}{16{,}000} = 2{\cdot}97.$$

Calculations of shear, adhesion, or compressive stress are of little value, as the beams failed in tension.

Tests No. 5.

Two arch-form beams or braces, 9 in. by 3 in. in section at midspan, tested at



midspan on a 9-ft. span. The arrangement of test is shown in *Figs.* 8 and 9. The cracks at failure are visible in *Fig.* 8. The results of tests are given in Tables 12 and 13.

TABLE 12.

Long Arch Beam. Weight 575 lbs.

Load lbs.	Deflection inches.	Load lbs.	Deflection inches.
280	0·02	3,280	0·17
1,780	0·04	3,780	0·20
1,280	0·06	4,280	0·23
	(crack at centre)	4,780	0·28
1,780	0·08	5,280	0·33
2,280	0·10	5,780	0·53
2,780	0·13	6,130 (*failed*)	

Deflection recovered when load released from 0·95 in. to 0·45 in.

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TABLE 13.

Short Arch Beam. Weight 415 lbs.

Load lbs.	Deflection inches.	Load lbs.	Deflection inches.
440	0·01	3,440	0·15
940	0·03	3,940	0·17
1,440	0·05	4,440	0·21
1,940	0·07 (crack at centre)	4,940	0·25
2,440	0·09	5,440	0·31
2,940	0·11	5,900 (*failed*)	

Analysis of Tests No. 5.

	Applied breaking loads. lbs.
Beam No. 1	6,130
,, No. 2	5,900
Sum	12,030
Mean	6,015

Weight of beam between supports

$$= 9 \text{ ft.} \times \frac{12 \times 3}{144} \times 150 = 337 \text{ lbs.}$$

Equivalent load at mid-span = 168 lbs.
Total breaking load at mid-span = 6,183 lbs.
Section at midspan = 9 in. × 3 in.
Depth to steel at midspan = 7½ in.
Radius arm = 6·6 in.
Area of steel (1-in. dia. rod) = 0·78 sq. in.
Bending moment at failure

$$= \frac{Wl}{4} = \frac{6{,}183 \times 108 \text{ in.}}{4}$$

$$= 167{,}000 \text{ in.-lbs.}$$

Tensile stress in steel at failure

$$= \frac{167{,}000}{6{\cdot}6 \times {\cdot}78} = 32{,}500 \text{ lbs. per sq. in.}$$

Factor of safety as against safe working stress of 16,000 lbs. per sq. in.

$$= \frac{32{,}500}{16{,}000} = 2{\cdot}03.$$

No test was made of the yield point of the steel, but there is little doubt it was about 32,500 lbs. per sq. in.

Tests No. 6.

Slabs 3 ft. 6 in. by 2 ft. 1 in. by 1½ in. thick. Tested on a 3-ft. span with a central load applied on a line across the slab at midspan. The results are given in

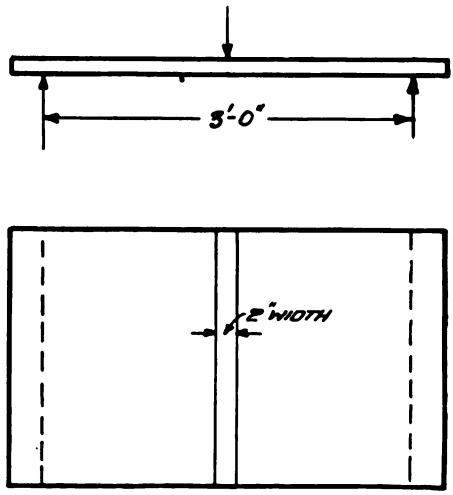

Table 14. (N.B.—Loaded all along centre line; supported at ends.)

TABLE 14.

(1) Maximum load 1,154 lbs. First crack noted 290 lbs. (possibly tested bottom side up).
(2) Maximum load over 1,780 lbs. First crack noted 1,000 lbs. Deflection ¼ in. at 1,500 lbs.
(3) Maximum load 1,680 lbs. First crack about 680 lbs. Deflection 1 in. at 1,500 lbs.
(4) Maximum load 1,800 lbs. First crack about 760 lbs.
(5) Load at centre on a block 6 in. by 8 in. cross section, 8-in. side, parallel to short side of slab supported as above. Maximum load 1,820 lbs. First crack about 760 lbs. This slab had double reinforcement of expanded metal. Other slabs had single reinforcement of expanded metal.

Analysis of Tests No. 6.

Discarding slab No. 1 (for reason given in Table 14) and No. 5 (which was a special test), the remainder give

	Breaking load. lbs.
Slab 2	1,780
,, 3	1,680
,, 4	1,800
Sum	5,260
Mean	1,753

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Weight of slab between supports

$$3 \text{ ft.} \times \frac{25 \times 1\frac{1}{2}}{144} \times 150 = 117 \text{ lbs.}$$

Equivalent point load at midspan = 58 lbs.

Total breaking load at midspan = 1,811 lbs.

Cross-sectional area of expanded metal

$$16 \times \tfrac{3}{8} \times \tfrac{1}{16} = 0{\cdot}375 \text{ sq. in.}$$

Depth to steel = $1\frac{1}{8}$ in.

Radius = 1 in.

Bending moment at failure

$$= \frac{Wl}{4} = \frac{1{,}811 \times 36 \text{ in.}}{4}$$

$$= 16{,}300 \text{ in.-lbs.}$$

Tensile stress in steel

$$f = \frac{16{,}300}{{\cdot}375 \times 1} = 43{,}500 \text{ lbs. per sq. in.}$$

Factor of safety on safe working stress of 16,000 lbs. per sq. in.

$$= \frac{43{,}500}{16{,}000} = 2{\cdot}72.$$

GENERAL CONCLUSIONS.

The results were consistent and uniform. This speaks equally well for the material, the design, and the accuracy of the testing.

The great strength of the cement does not come out from the tests (except those of the test cubes), as the steel always failed first. This was inevitable, as the beams had to carry their load when two days old and the concrete went on increasing in strength before the date of testing; naturally the steel did not, and was therefore the weaker element at the time of test. Therefore the test results were determined by the steel, and it is satisfactory to note the close concordance between actual results and those calculated by the writer and communicated to the Company previous to the tests.

The way in which the specimens continued to carry their loads with excessive deflections long after cracking is also a valuable feature of the specimens tested.

The writer wishes to acknowledge the great assistance he has received from the staff of the Civil Engineer Department of the City and Guilds (Engineering) College, especially Professor S. M. Dixon.

"The Foreman's Guide to Concrete."

Owing to pressure on our space we are unable to give in this issue Part V of Mr. Albert Lakeman's series of articles entitled "The Foreman's Guide to Concrete." We hope to continue these in our next issue.

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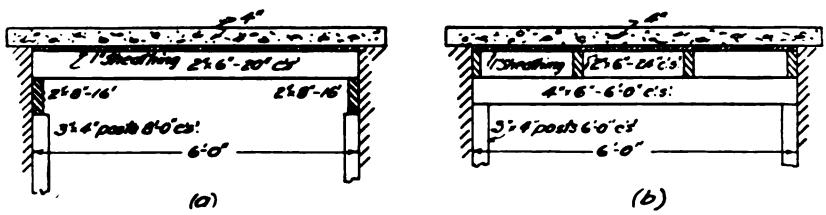
DESIGN OF FORMWORK FOR REINFORCED CONCRETE CONSTRUCTION.

By A. E. WYNN, B.Sc., A.M.Am.Soc.C.E.

V.—DESIGN PROBLEMS.

DESIGN 1: SIMPLE SLAB.

Design forms for a 4-in. corridor slab, span 6 ft., supported on brick walls.



DESIGN-1

(*a*) **Joists Spanning across Slab.**—From Table 1, 1-in. sheathing will span up to $31\frac{1}{2}$ in.; from Table 2, 2 in. by 6 in. joists at 20 in. on centre can be used; so strength of joists will govern spacing.

The joists will be supported at their ends by ledgers along the wall. The load carried by each joist to ledger will be $(48 + 40)\ 1{\cdot}67 \times 3 = 440$ lbs. From Table 5, we have the choice of 2 in. by 6 in. spanning 6 ft., 2 in. by 8 in. spanning 8 ft., or 3 in. by 6 in. spanning 7 ft. 6 in., or 4 in. by 6 in. spanning 8 ft. Assuming 3 in. by 4 in. posts, the 2 in. by 8 in. will give the least amount of timber and will be used.

Maximum load on post (calculated from formula 26) with one joist over post = 2130 lbs. The 4-in. side of the post will be placed against the wall. Bearing required for ledger $= \dfrac{2130}{400 \times 1{\cdot}75} = 3$ in., and we have $3\frac{3}{4}$ in. From Table 11 it will be seen that a 3 in. by 4 in. post is sufficiently strong.

(*b*) **Longitudinal Joists.**—If it is desired to run the joists longitudinally they can be designed as continuous and the spacing of the joists may be increased 25 per cent. With one centre joist the span of the sheathing would be 36 in., which is too great for 1-in. sheathing, so two centre joists will be used. The spacing will then be 24 in., and from Table 4 for 2 in. by 6 in. joists and 4-in. slab, adding 25 per cent. to spacing, the span or distance between ledgers can be 6 ft.

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Load from each joist on ledger = 88 × 2 × 6 = 1056 lbs., and from Table 5, for a span of 6 ft. and joists 24 in. centre to centre a 4 in. by 6 in., made up of two 2 in. by 6 in., can just be used.

Load on post will be 1056 × 1·5 = 1584 lbs. This being less than before no further calculations are necessary.

Comparing the relative economy in timber, we have for one lin. ft. of slab in cu. ft. of timber, assuming posts 10 ft. long:

(*a*) sheathing	$\frac{1}{12} \times 6$	= 0·5	(*b*)		= 0·5
joists	$\frac{12 \times 6 \times 12}{144 \times 20}$	= 0·3		$\frac{12 \times 4}{144}$	= 0·333
ledgers	$\frac{16 \times 2}{144}$	= 0·222		$\frac{24 \times 6}{144 \times 6}$	= 0·167
posts	$\frac{12 \times 10 \times 2}{144 \times 8}$	= 0·208		$\frac{12 \times 10 \times 2}{144 \times 6}$	= 0·278
		= 1·230 c.f.			= 1·278 c.f.

It will be seen that there is little difference in the amount of timber required in the two designs. The second design would be slightly better perhaps because only one size of timber is required and the joists can be used in long lengths from which there would be better salvage than if they were cut up into 6 ft. lengths, while cutting the sheathing would not matter much since short lengths can always be used.

DESIGN 2: SIMPLE SLAB, LONG SPAN.

Design forms for a 7 in. roof slab 14 ft. by 20 ft., 10 ft. high.

A span of 14 ft. will require one or two rows of posts to reduce the span.

(*a*) **One Row of Posts.**—Joists will be continuous, so 25 per cent. can be added to the spacing. From Table 4, for 7 ft. span and 7 in. slab, 2 in. by 8 in. joists at 21 × 1·25 = 26 in. c.s. can be used. From Table 1, 1-in. sheathing will span up to 29½ in.

Load from each joist on ledger = (84 + 40) × 2·17 × 7 = 1885 lbs.

From Table 5 we have choice of 2 in. by 8 in., span 4 ft. 6 in.; or 4 in. by 8 in., span 6 ft. The former will give less timber per lin. ft., so will be used.

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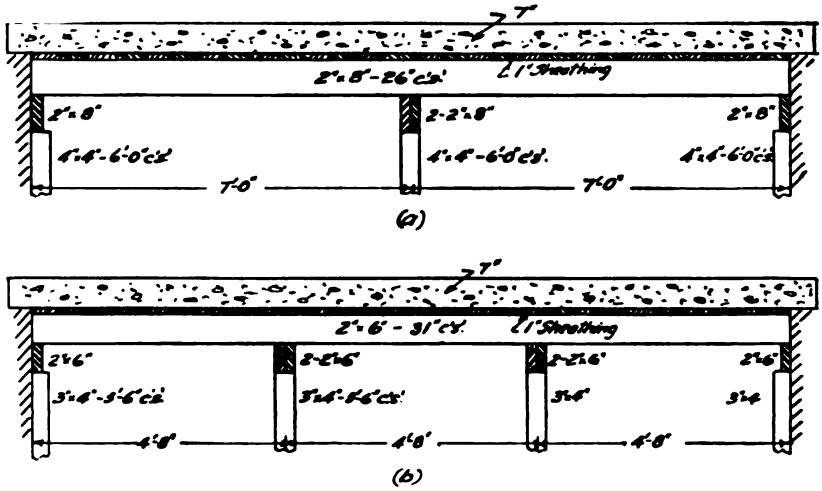
Maximum load on post with one joist over post = 1885 (1 + 2 (·52 + ·037)) = 4000 lbs.

Bearing stress on ledger = $\frac{4000}{1{\cdot}75 \times 3{\cdot}75}$ = 600 lbs. sq. in., which is too high and indicates the ledger is too narrow, so a 4 in. by 8 in., made up of two 2 in. by 8 in., must be used.



Load on post now = 1885(1 + 2(·64 + ·28)) = 5350 lbs., and bearing stress = $\frac{5350}{3{\cdot}75 \times 3{\cdot}75}$ = 380 lbs. sq. in., which is safe.

Load on wall ledgers = half the above = 942 lbs.

From Table 5 one 2 in. by 8 in. will carry the load at 6 ft. span.

Wall posts will be the same as interior posts for convenience. Bearing stress on wall ledger = $\frac{2675}{1{\cdot}75 \times 3{\cdot}75}$ = 407 lbs. sq. in., or just at the limit of the allowable stress.

(*b*) **Two Rows of Posts.**—If it is desired to use smaller joists and ledgers, use two rows of posts. Span now is 4 ft. 8 in. In Table 4 interpolating between 4 ft. 6 in. and 5 ft. spans, 2 in. by 6 in. at 27 × 1·25 = 34 in. can be used. But from Table 1 maximum span of 1-in. sheathing = 29½ in. and adding 1½ in. for width of joist gives 31 in. for maximum spacing of joists.

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Loads on ledger $= 124 \times 2{\cdot}58 \times 4{\cdot}67 = 1500$ lbs.

From Table 5, using same size timber, 4 in. by 6 in. (two 2 in. by 6 in.) will carry the loads at 5 ft. 6 in. span.

Load on post $= 1500(1 + 2({\cdot}53 + {\cdot}06)) = 3270$ lbs. From Table 10 a 3 in. by 4 in. post will do, checking bearing stress,

$$\text{stress} = \frac{3270}{3{\cdot}75 \times 2{\cdot}75} = 318.$$

Loads on wall ledger $= 750$, and one 2 in. by 6 in. will carry the loads at the same span of 5 ft. 6 in.

Comparing the two designs for economy of material, in cu. ft. of timber per lin. ft. of slab, assuming 1 in. × 6 in. post braces each way.

(*a*) Sheathing	$\frac{14}{12}$	$= 1{\cdot}17$	(*b*) $\frac{14}{12}$	$= 1{\cdot}17$
Joists	$\frac{16 \times 12 \times 14}{144 \times 26}$	$= 0{\cdot}72$	$\frac{12 \times 12 \times 14}{144 \times 31}$	$= 0{\cdot}45$
Ledgers	$\frac{64}{144}$	$= 0{\cdot}45$	$\frac{72}{144}$	$= 0{\cdot}50$
Posts	$\frac{16 \times 30}{144 \times 6}$	$= 0{\cdot}56$	$\frac{12 \times 40}{144 \times 5{\cdot}5}$	$= 0{\cdot}61$
Braces	$\frac{6 \times 32}{144 \times 6}$	$= 0{\cdot}22$	$\frac{6 \times 36}{144 \times 5{\cdot}5}$	$= 0{\cdot}27$
		3·12 c.f.		3·00 c.f.

There is not much difference between the two designs for amount of material used, so it will be a question of the relative cost of sizes.

DESIGN 3: BEAM AND GIRDER PANEL, SHORT SPAN SLAB.

Design forms for typical bay of beam and girder construction, consisting of 3½-in. slab, 8 in. by 18 in. beams, span 22 ft. and 5 ft. 4 in. on centres and 12 in. by 20 in. girders, span 16 ft.

From Table 1, 1-in. sheathing for 3½-in. slab will span 32 in.

Span of joists will be 4 ft. 6 in. From Table 4, interpolating between 3 in. and 4 in. slabs, we can use 2 in. by 4 in. at 16½ in. on centre, or 2 in. by 6 in. at 39 in. on centre, which, however, must be reduced to $32 + 1 = 33$ in. on centre because of the deflection of the sheathing. The latter will be more economical, so will be chosen.

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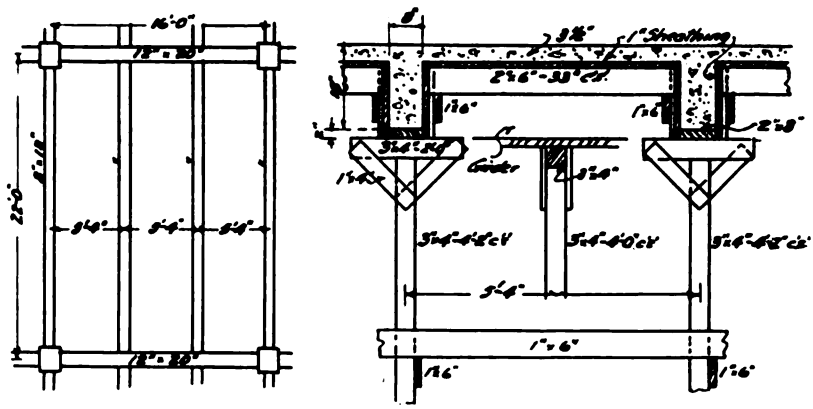
Load carried to ledger, including 40 lbs. live load $= (42 + 40) \times 2{\cdot}75 \times 2{\cdot}25 = 510$ lbs.

Width of ledger required for bearing $= \dfrac{510}{400 \times 1{\cdot}75} = {\cdot}73$ in., so 1 in. by 6 in. can be used.

Beam bottoms will be 2 in. by 8 in., and according to our rule a 16-in.



DESIGN-3

depth requires posts 4 ft. 3 in. on centre and a 24 in. depth 4 ft. on centre; since the clear span of the beams is 21 ft., it will be satisfactory to space the posts 4 ft. 2 in. on centre.

The girder bottom will be 2 in. by 12 in., made out of two 2 in. by 6 in. The spacing of the posts under the girders may be 4 ft.

Load on beam posts

$$= (82 \times 5{\cdot}33 \times 4{\cdot}17) + (8 \times 14{\cdot}5 \times 4{\cdot}17) = 2305 \text{ lbs.}$$

Load on girder posts

$$= (82 \times 5{\cdot}17 \times 4) + (12 \times 16{\cdot}5 \times 4) + (8 \times 14{\cdot}5 \times 4{\cdot}17) = 2975 \text{ lbs.}$$

From table 11, 3 in. by 4 in. posts can be used.

DESIGN 4: BEAM AND GIRDER PANEL, LONG SPAN SLAB.

Design forms for typical beam and girder panel consisting of 4½ in. slab with 1 in. finish of cement, with 12 in. by 24 in. beams spanning 30 ft. and 14 in. by 30 in. girders spanning 24 ft.

Total thickness of slab will be 5½ in. From Table 1, 1-in. sheathing will span 30½ in. Clear span of joists is 11 ft. To span this without a centre support would mean using heavy joists close together, so one

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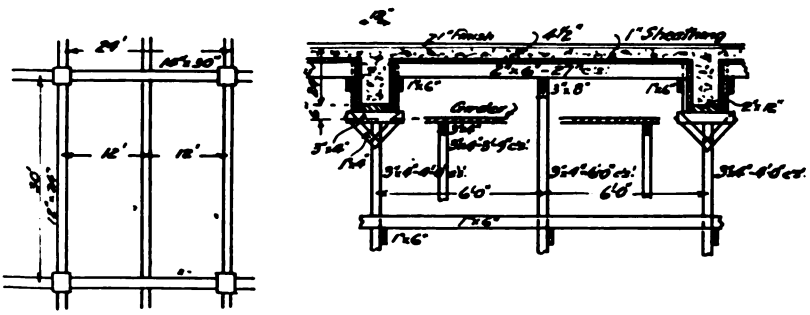
centre ledger will be used, reducing the span to 5 ft. 6 in., and joists will be continuous, so that 25 per cent. can be added to the spacing.

Interpolating between 5 in. and 6 in. slabs, and adding 25 per cent., we can use 2 in. by 6 in. at 27 in. on centre.

Loads on centre ledger = $(66 + 40)2{\cdot}25 \times 5{\cdot}5 = 1315$ lbs. From Table 5 we have choice of 2 in. by 8 in. span 5 ft., 4 in. by 6 in.



DESIGN-4

span 5 ft. 6 in., 3 in. by 8 in. span 6 ft., 4 in. by 8 in. span 7 ft. 6 in. For economy in material they are approximately in the ratio of 1 : 1·1 : 1·05 : 1·1. Available material and relative cost will decide ; we will use 3 in. by 8 in. span 6 ft.

Loads on beam ledger will be 658 lbs., and we can tell from previous design that a 1 in. by 6 in. will be sufficient.

Maximum load on ledger posts = $1315(1 + 2(\cdot 625 + \cdot 25)) = 3025$ lbs.

Bearing stress on 4 in. post = $3025/3{\cdot}75 \times 2{\cdot}75 = 293$, so we can use 3 in. by 4 in. posts.

To show the use of the formulæ we will investigate shear and deflection of the ledger.

With one joist 4 in. from centre of post

$$V = 1315\frac{(5{\cdot}67 + 3{\cdot}42 + 1{\cdot}17)}{6} = 2250$$

and unit shearing stress $v = \dfrac{1{\cdot}5 \times 2250}{2{\cdot}75 \times 7{\cdot}75} = 159$ lbs. sq. in., which is well within the allowable.

Deflection (formula 32),

$$D = \frac{5 \times 1315 \times 12}{384 \times 1200000 \times 2{\cdot}75 \times 7{\cdot}75^3} \times (72^3 + 2 \times 9(3 \times 72^2 - 4 \times 9^2))$$

$= 0{\cdot}087$ in. which is less than $\frac{1}{8}$ in.

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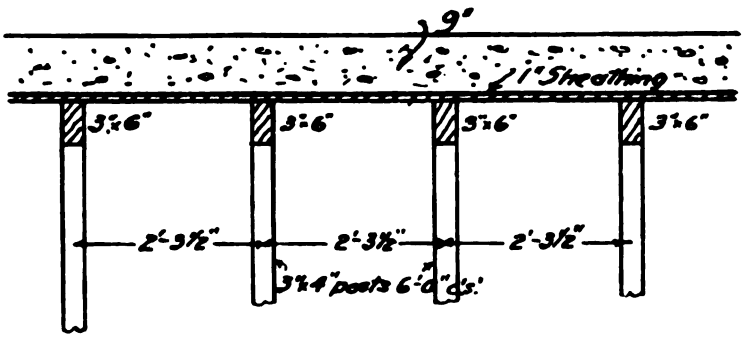
Beam bottoms will be 2 in. thick, and the spacing of the posts 4 ft. Girder bottoms will be 2 in. thick, and the spacing of the posts will be 3 ft. 9 in.

DESIGN 5: MUSHROOM SLAB, ONE-WAY DESIGN.

Design forms to support a 9 in. flat floor slab without beams; floor height 11 ft.



DESIGN 5

This design can be used for any large area of floor without beams.

From Table 1, 1 in. sheathing will span 28½ in.

From Table 4, continuous joists 3 in. by 6 in. spanning 6 ft. can be placed 27½ in. on centre.

Load on posts = (108 + 40) × 2·29 × 6 = 2030 lbs. Use 3 in. by 4 in. posts.

DESIGN 6: MUSHROOM SLAB, TWO-WAY DESIGN.

Design forms for a 9 in. slab as before, using joists and ledgers.

Instead of carrying the joists individually on posts, ledgers may be used with smaller joists.

From Table 4 (continuous joists), 2 in. by 6 in. at 25 in. on centre will span 5 ft.

Loads on ledgers from joists = 148 × 2 08 × 5 = 1540 lbs.

From Table 5, the best ledgers to use are 4 in. by 6 in. span 5 ft., or 3 in. by 8 in. span 5 ft. 6 in.; we will use the latter as it takes less timber.

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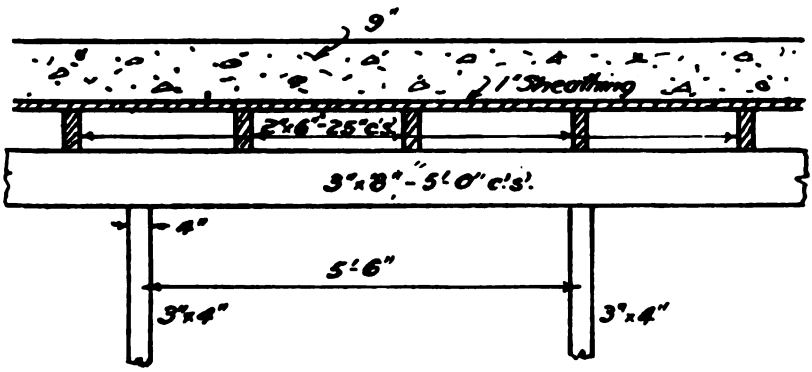
Maximum load on posts $= 1540 \times (1 + 2(\cdot 621 + \cdot 243)) = 4200$ lbs.

Bearing stress on post 4 in. wide $= \dfrac{4200}{3\cdot 75 \times 2\cdot 75} = 407$, so that it is just possible to use a 3 in. by 4 in. post, with the long side parallel to the ledger.



DESIGN 6

Comparing the two designs 5 and 6 for economy in timber, in cu. ft. of timber per sq. ft. of floor,

(5) sheathing		$= 0\cdot 083$ c.f.	(6)	$= 0\cdot 083$
joists	$\dfrac{18 \times 12}{144 \times 27\cdot 5}$	$= 0\cdot 055$	$\dfrac{12 \times 12}{144 \times 25}$	$= 0\cdot 040$
posts	$\dfrac{12 \times 10}{144 \times 2\cdot 29 \times 6}$	$= 0\cdot 061$	$\dfrac{120 \times 1}{144 \times 5\cdot 5 \times 5}$	$= 0\cdot 030$
braces	$\dfrac{6 \times 8\cdot 29}{144 \times 2\cdot 29 \times 6}$	$= 0\cdot 025$	$\dfrac{6 \times 10\cdot 5}{144 \times 27\cdot 5}$	$= 0\cdot 016$
		ledger	$\dfrac{24 \times 1}{144 \times 5\cdot 5}$	$= 0\cdot 030$
		$= 0\cdot 224$ c.f.		$= 0\cdot 199$ c.f.

This shows that it is more economical to use the two-way system, saving about 10 per cent. of the timber required by the one-way. This holds good in general, and for that reason is more popular.

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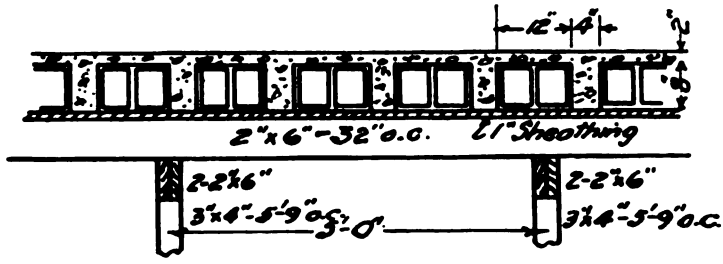
DESIGN 7: LIGHT WEIGHT RIB FLOORS.

By this is meant that system of floor construction which consists of clay, metal, or gypsum tile fillers, combined with narrow concrete joists of long span, supporting a thin slab; the slab may be omitted.

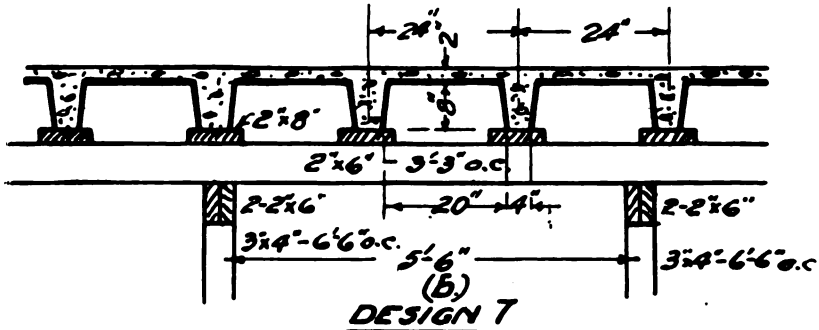
(*a*) **Closed Deck.**—Clay tile fillers are usually 12 in. wide, combined with 4 in. wide joists of variable depth depending on the span carrying a 2 in. slab.

Since the tile is narrow and the joists close together, with tongue and groove sheathing, the heavier weight of the joist is so distributed that the load may be considered uniform, and the deck is often built solid as for a solid slab.

Design forms for a floor 10 *in. deep, consisting of* 4 *in. joists with* 2 *in. slab and* 12 *in. wide by* 8 *in. deep clay tile fillers.*



(a.)



(b.)

DESIGN 7

An 8 in. by 12 in. by 12 in. tile weighs 30 lbs., the slab 25 lbs., and the joist 40 lbs., or total of 95 lbs. per foot of joist, or a uniform load of 95/1·33 = 70 lbs. per sq. ft., which is equivalent to a 5½ in. solid slab. From Table 1. maximum span of 1 in. sheathing is 30½ in.; adding 1½ in. for joist, joists will be 32 in. on centre.

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From Table 4, adding 25 per cent. to spacing, for spacing of 32 in. with $5\frac{1}{2}$ in. slab, 2 in. by 6 in. joists will span 5 ft.

Loads carried by joists to ledger $= (70 + 40) \times 2{\cdot}67 \times 5 = 1470$ lbs.

From Table 5, 4 in. by 6 in. ledgers will span 5 ft. 9 in. with these loads.

Load on posts $= 1470(1 + 2(0{\cdot}535 + 0{\cdot}073)) = 3250$ lbs., and a 3 in. by 4 in. will be sufficient.

(*b*) **Open Deck.**—Metal tiles are generally about 20 in. wide at the bottom and are much lighter than clay tiles, so that the load is concentrated at the joists; these forms are therefore built with a 2 in. plank under the joist, the rest of the deck being left open.

Design forms for a floor consisting of 4 in. wide joists 8 in. deep, 24 in. on centre carrying a 2 in. slab, with metal tile fillers.

Tiles will weigh about 3 lbs. each, joist 36 lbs., slab 48 lbs., or, say, a total load of 90 lbs. per foot of joist. Live load per foot $= 75 \times 2 = 150$, or total load of 240 lbs. per foot of joist.

Using 2 in. by 8 in. under joists the maximum span is found from formula 14.

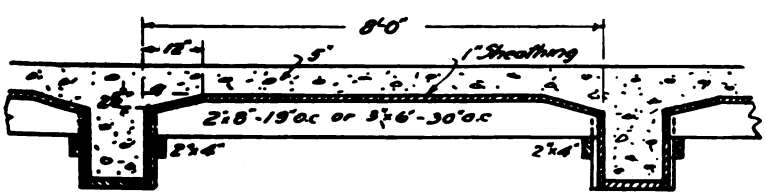
$$\text{for } D = 1/8,\ \frac{I}{8} = \frac{3 \times 340 \times 1^4 \times 12^4 \times 12}{384 \times 12 \times 1200000 \times 7{\cdot}5 \times 1{\cdot}75^3} \text{ or } 1 = 3 \text{ ft. } 3 \text{ in.}$$

Loads on joist $= (90 + 80) \times 3{\cdot}25 = 555$ lbs., and from Table 5 a 2 in. by 6 in. will span 5 ft. 6 in.

If desired to use ledgers at 5 ft. 6 in. on centre instead of shoring each joist a 4 in. by 6 in. will span 6 ft. 6 in. with the load of 1565 lbs. per joist. Use 3 in. by 4 in. posts.

DESIGN 8: FLOOR SLABS WITH SMALL HAUNCHES AT THE BEAMS.

Design forms for a 5 in. slab, span 8 ft. between beams, with haunches $2\frac{1}{2}$ in. deep and 12 in. long at each beam.



DESIGN 8

When the haunch is small the joists can be cut to suit the haunch.

From Table 1, 1 in. sheathing will span 31 in.; from Table 2 we can use 2 in. by 8 in. at 19 in., or 3 in. by 6 in. at 30 in.

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As joists will be cut away at the ends the horizontal shearing stress must be investigated, otherwise the design is the same as before.

With 2 in. by 8 in.,

$$v = \frac{1{\cdot}5 \times \{(60 + 40) \times 1{\cdot}58 \times 4 + 15 \times 1{\cdot}58\}}{1{\cdot}75 \times 5{\cdot}25} = 107 \text{ lbs. sq. in.}$$

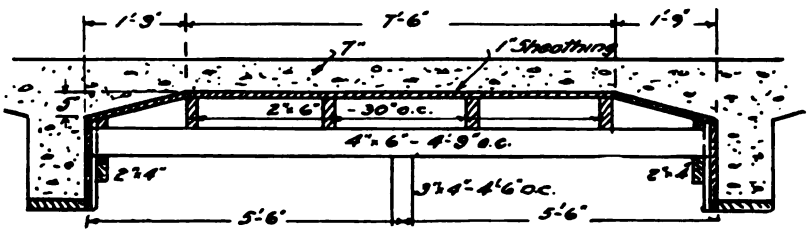
With 3 in. by 6 in.,

$$v = \frac{1{\cdot}5 \times \{100 \times 2{\cdot}5 \times 4 + 15 \times 2{\cdot}5\}}{2{\cdot}75 \times 3{\cdot}25} = 174 \text{ lbs. sq. in.}$$

So that either size joist may be used.

DESIGN 9: FLOOR SLABS WITH DEEP HAUNCHES AT THE BEAMS.

Design forms for a 7 in. slab span 11 ft. with a haunch 5 in. deep and 1 ft. 9 in. long at the beams.



DESIGN 9

When the haunch is deep it is often better to run the joists longitudinally and carry them on ledgers supported by the beams and so avoid the waste of cutting.

The haunch sheathing can span from beam side to the first joist at the end of the haunch; the average slab thickness will be $9\frac{1}{2}$ in. and 1 in. sheathing will be sufficient.

Depth of joist should be about the same as that of haunch (not less), and will be continuous.

From Table 4, 2 in. by 6 in. at $26 \times 1{\cdot}25 = 32\frac{1}{2}$ in. will span 4 ft. 9 in.

From Table 1, 1 in. sheathing will span $29\frac{1}{2} + 1\frac{1}{2} = 31$ in., so this governs; however, since total width of slab is 7 ft. 6 in. we will use 3 spaces at 30 in.

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Loads on ledger (assuming same for all joists) = (84 + 40) × 2·5 × 4·75 = 1475 lbs. The span of 11 ft. will be too great without a centre post. With centre row of posts, from Table 5 a 4 in. by 6 in. will carry the load over a span of 5 ft. 6 in.

Shear at end of ledger (approx.)

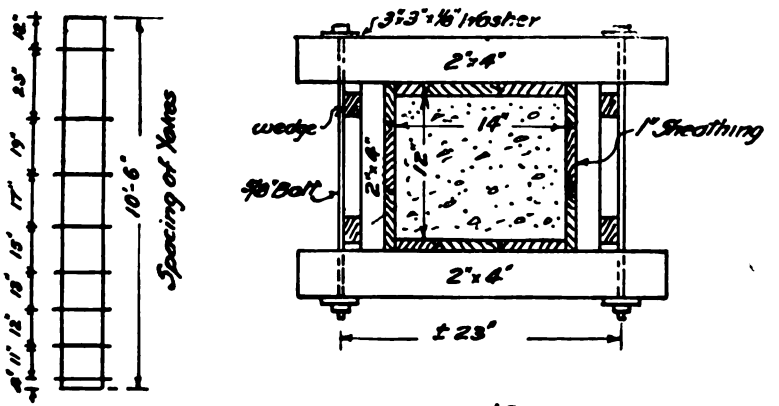
$$= 290 \times 4{\cdot}75 \times \frac{3{\cdot}75}{5{\cdot}5} + 310 \times 4{\cdot}75 \times \frac{1{\cdot}25}{5{\cdot}5} = 1235,$$

width of beam ledger $= \dfrac{1235}{1{\cdot}75 \times 400} = 1{\cdot}76$; use 2 in. by 4 in.

Load on centre posts will be 3160 lbs.; a 3 in. by 4 in. can be used.

DESIGN 10: SMALL COLUMN FORM.

Design form for a column 12 in. by 14 in., 10 ft. 6 in. high.



DESIGN 10.

The form will not be used over twice, so 1 in. sheathing will be used.

From Table 6, a line drawn through depth of 10 ft. 6 in. will give the first spacing for a 14 in. column at 11 in. Since the first yoke should be about 4 in. up from the bottom, the spacing will be 4 in., 11 in., 12 in., 13 in., 15 in., 17 in., 19 in., 23 in. This will give the top yoke 12 in. below the top of the column. Yokes spanning the 12 in. dimension can be 2 in. by 4 in. on the side instead of edge.

To show the size of bolts and washers theoretically required, they will be calculated from formulæ 43, 44 and 45.

Maximum stress in bolts will be in the second yoke up, and diameter of bolt $d = \sqrt{\dfrac{9{\cdot}25 \times {\cdot}96 \times 1{\cdot}17}{175}} = 0{\cdot}243$ in., say $\frac{1}{4}$ in.

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As this is a small column a $\frac{1}{4}$ in. bolt could be used, though a $\frac{5}{8}$ in. bolt would be better, as mentioned before.

For $\frac{1}{4}$ in. bolt, hole in washer will be $\frac{3}{8}$ in. diameter.

$$\text{Area of square washer} = \frac{125 \times 9{\cdot}25 \times {\cdot}96 \times 1{\cdot}17}{800} + 0{\cdot}785 \times 0{\cdot}375^2$$

$$= 1{\cdot}733 \text{ sq. ins., or side} = \text{say } 1\tfrac{1}{2} \text{ in.}$$

$$\text{Thickness of W.I. washer} = \frac{1}{220}\sqrt{\frac{400 \times (2{\cdot}25 - 0{\cdot}44^2) \times 0{\cdot}53}{0{\cdot}44}}$$

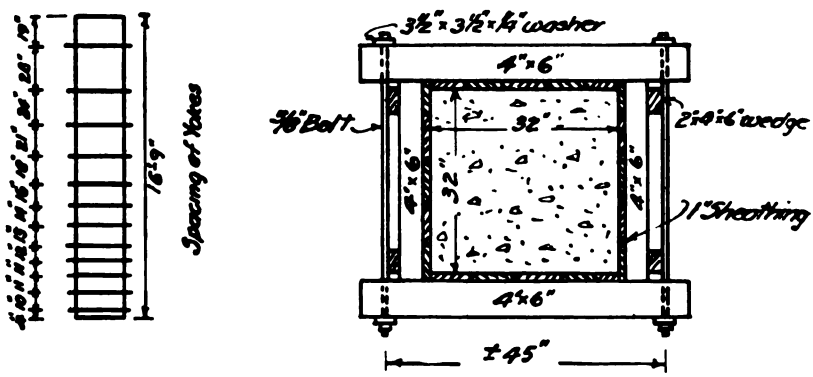
$$= 0{\cdot}143 \text{ in.}$$

$$= \text{say, } 5/32 \text{ in.}$$

It would be better to standardise on a larger size according to the rule given in Part 3 and use a $\frac{5}{8}$ in. bolt with a 3 in. by 3 in. by $\frac{1}{8}$ in. washer.

DESIGN 11: LARGE COLUMN FORM.

Design form for a 32 in. square column, 16 ft. 9 in. high.



DESIGN 11.

It will be seen from Tables 6 to 9 that a 4 in. by 6 in. yoke should be used. According to the Table, the first yoke will be 2 in. above the bottom of the column. Change this to 4 in. and space as given, 10 in., 11 in., 11 in., 12 in., etc., using $1\frac{1}{4}$ in. sheathing. Top yoke will be 19 in. from top of column. The other pair of yokes may be 4 in. by 6 in. on side instead of edge. Bolts will be $\frac{5}{8}$ in. diameter and washers $3\frac{1}{2}$ in. by $3\frac{1}{2}$ in. by $\frac{1}{4}$ in.

(*To be continued.*)

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Best British Portland Cement	per ton 58s. to	63	0
"Ferrocrete" Portland Cement	delivered London 10s. per ton extra		
"Super-Cement"	d/d 4 miles Charing Cross, per ton 88s. to	93	0

BOARDING FOR SHUTTERING—		Sawn. s.	Sawn. d.	Wrot. s.	Wrot. d.
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—	
3 in. by 4 in.	from £24 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £23 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	0
9/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds	,,	14	6
¼ in. Rounds	,,	15	6

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED WITH CLEAN WATER IN PROPORTION 1 : 2 : 4—

			s.	d.
Do.	do.	in foundation	per yard cube	48	6
Do.	do.	in columns	,, ,,	53	6
Do.	do.	in beams	,, ,,	53	6
Do.	do.	in floor slabs 4 in. thick	per yard super	5	8
Do.	do.	in floor slabs 6 in. thick	,, ,,	8	5
Do.	do.	in floor slabs 9 in. thick	,, ,,	12	3
Do.	do.	in walls 6 in. thick	,, ,,	8	8

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—

		s.	d.
From ¼ in. to ⅝ in.	per cwt.	25	0
,, 9/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2⅞ in.	,,	23	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1½*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2½*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.*

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.*

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	,, ,,	1	1½
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1½
Do. do. as last, in narrow widths	,, ,,	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Carpenters working on old shuttering, 1/10½; Labourers on building works, 1/4½; Operators on concrete mixers and hoists, 1/5½; Barbenders, 1/5½.

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PROSPECTIVE NEW CONCRETE WORK.

ALSTON.—*Bridge.*—The R.D.C. is considering a scheme for the erection of a bridge at Gossip Gate.

BALLYMENA. — *Houses.* — The R.D.C. has under consideration a scheme for the erection of 200 houses.

BEDLINGTON. — *Houses.* — The U.D.C. has approached the M.H. for permission to erect 78 houses.

BELLEEK.—*Bridges.*—The Fermanagh C.C. proposes to spend £64,000 on the construction of three bridges.

BIRKENHEAD.—*Houses.*—The Corporation has applied to the M.H. for sanction to a loan of £52,573 for erecting houses at the Deel and at Rock Ferry.

BLOXWICH.—*Road.*—The Walsall T.C. proposes to construct a road from Four Crosses to High Street, at a cost of about £14,249.

BOURNEMOUTH.—*Sea Wall.*—The T.C. is considering a scheme for the construction of a sea wall at Fisherman's Walk.

BOVEY TRACEY.—*Houses.*—The Parish Council has asked the Newton Abbot R.D.C. to erect 50 houses at Bovey Tracey.

BROADWAY (WORCS.). — *Reservoir.* — The T.C. has approached the R.D.C. with a view to providing the parish with a 250,000-gallon reservoir.

BURTON.—*Footbridge.*—A scheme is being considered for the construction of a footbridge over the Barnlake Ferry, estimated to cost £3,000.

CASTLE DOUGLAS.—*Sewage Works.*—The T.C. proposes to construct new sewage disposal works.

CHELMSFORD.—*Houses.*—The T.C. proposes to build 100 houses on the Boarded Barns estate.

COVENTRY. — *Concrete Houses.* — The City Council proposes to erect 2 experimental concrete houses at a cost of £1,000.

DURRINGTON. — *Houses.* — The East Preston R.D.C. is considering the construction of 52 houses.

EARITH.—*Concrete Bridge.*—The Isle of Ely and Huntingdonshire County Councils are considering a proposal to erect a reinforced concrete bridge in place of the present bridge at Earith, at an estimated cost of £8,000.

ERPINGHAM. — *Houses.* — The R.D.C. has applied to the M.H. for a loan of £60,000 for housing purposes.

EVINGTON. — *Sewage Works.* — The Billesden R.D.C. has applied for sanction to a loan of £9,100 for sewage disposal works.

GODSTONE.—*Houses.*—The R.D.C. proposes to erect 200 houses.

GREAT YARMOUTH. — *Concrete Roads.* —The Corporation proposes to lay a concrete road from Salmon Road to the White House, at an estimated cost of £9,150, and to replace the granite setts in George Street with concrete, at a cost of £2,311.

HALFWAY.—*Houses.*—The Chesterfield R.D.C. is considering the question of building 200 houses at Halfway.

HASLINGDEN.—*Houses.*—The T.C. has applied for sanction to borrow £21,300 for the construction of 32 houses.

HULL.—*Concrete Houses.*—The Corporation proposes to erect 200 concrete houses by direct labour.

LANGLAND BAY.—*Retaining Wall.*—The Swansea Corporation proposes to construct a concrete retaining wall across the back of the bay, at an estimated cost of £5,000.

LEXDEN.—*Houses.*—The R.D.C. has applied for permission to erect 124 houses.

LIMERICK.—*Roads.*—The Corporation proposes to re-pave a number of roads in concrete, at a cost of about £39,000.

MARKET BOSWORTH.—*Houses.*—The R.D.C. proposes to erect 250 houses.

MOLESEY.—*Houses.*—The R.D.C. is considering a scheme for the erection of 50 houses.

NEWCASTLE-ON-TYNE. — *Houses.* — The Corporation Housing Committee has applied for permission to build 50 houses at Pendower.

NUNEATON.—*Houses.*—The T.C. has approached the M.H. for sanction to erect 250 houses.

ROTHERHAM.—*Houses.* — The R.D.C. proposes to erect 500 houses.

ROYSTON.—*Houses.*—The U.D.C. has applied for a loan of £27,370 for the construction of 56 houses.

RUGBY.—*Houses.*—The U.D.C. proposes to erect 100 houses on the Eastlands Estate.

SHEFFIELD.—*Reservoir.*—The Hoyland Nether U.D.C. proposes to construct a reinforced concrete water tower and reservoir, at an estimated cost of £38,000.

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SUNDERLAND.—*Bridge.*—The Corporation proposes to build an 80-ft. bridge in place of the existing Wearmouth Bridge.

TAUNTON.—*Concrete Houses.* — The T.C. proposes to erect a number of concrete cottages by direct labour.

THORNABY.—*Houses.*—The T.C. is considering a scheme for the erection of 50 houses.

TORQUAY.—*Sea Defence Work.*—The T.C. proposes to carry out sea defence work, estimated to cost £32,000.

WESTON-SUPER-MARE. — *Road.* — The U.D.C. proposes to lay a road from Milton Road to Bristol Road.

WHITEHAVEN.—*Road.*—The T.C. proposes to lay a road from Victoria Road to the Loop Road.

WILLINGTON. — *Houses.* — The U.D.C. has a scheme on hand for the construction of 60 houses.

WIMBORNE. — *Houses.* — The U.D.C. has applied to the M.H. for sanction to erect 40 houses.

TENDERS ACCEPTED.

BIRMINGHAM. —*Concrete Houses.* —The T.C. has accepted the tender of Mr. M. A. Boswell, Wolverhampton, at £425 each, for the erection of 1,300 concrete houses.

BOSTON (LINCS.).—*Concrete Bridge.*—The Lincolnshire C.C. has accepted the tender of Messrs. W. Wright & Son (Lincoln), Ltd., for the construction of a reinforced concrete bridge across the new Hammond Beck.

CARDIFF.—*Steel Concrete Pipes.*—The T.C. has accepted the tender of Messrs. Guest, Keen & Nettlefold, at £109,561, for the supply of steel-lined concrete pipes.

EASTBOURNE. — *Concrete Houses.* — The T.C. recommends the acceptance of Messrs. Henry Boot & Sons' tender, at £560 each, for the erection of 200 concrete houses.

ENNISKILLEN. — *Concrete Bridges.*—The County Council has accepted the tender of Mr. S. R. Boyd, Belfast, at £43,000, for the construction of three reinforced concrete bridges.

HERTFORD. — *Concrete Houses.* — The Corporation has accepted the tender of Messrs. Botsford, Vale and Wightman, Hertford, at £12,000, for the construction of 24 concrete houses.

HOVE. — *Pier.* — Messrs. Armstrong, Whitworth & Co. have secured the contract for the construction of a reinforced concrete pier, estimated to cost over £200,000.

HULL.—*Concrete Houses.*—The T.C. has accepted the tender of Messrs. Kettlewell, Son & Co., at £475 per house, for the erection of 6 concrete houses.

LIVERPOOL. — *Concrete Houses.* — The Corporation has accepted the tender of Mr. M. A. Boswell, Wolverhampton, for the construction of 500 concrete houses.

LIVERPOOL. — *Concrete Houses.* — The Corporation has accepted the tender of the Bonding Block Construction Co., Ltd., for the construction of 50 concrete houses.

LOCHEND. — *Concrete Houses.* — The Edinburgh T.C. has accepted the tender of the Corolite Construction Co., at £420 each, for the erection of 52 concrete houses.

NORTHWICH. — *Water Tower.* — The R.D.C. has accepted the tenders of Messrs. S. & J. Smethurst, Ltd., at £2,514 14*s.* 10*d.*, for the construction of a 300,000-gallon reinforced concrete reservoir; and Mr. W. H. Brocklesby, Birkenhead, at £2,500 6*s.* 2*d.*, for the construction of a 120,000-gallon reinforced concrete water tower.

ROTHERHAM. — *Sewage Works.* — The R.D.C. has accepted the tender of Messrs. Mollekin & Sons, at £12,242, for the construction of sewage disposal works at Brampton Bierlow.

Trade Notice.

WITH reference to the photograph of the retaining wall at Viscount Leverhulme's residence at Hampstead, published in an advertisement by the British Reinforced Concrete Engineering Co., Ltd., in our last issue, the Company wish us to state that the architectural work has been carried out by Messrs. T. H. Mawson & Sons, in conjunction with Mr. Leslie Mansfield, F.R.I.B.A.

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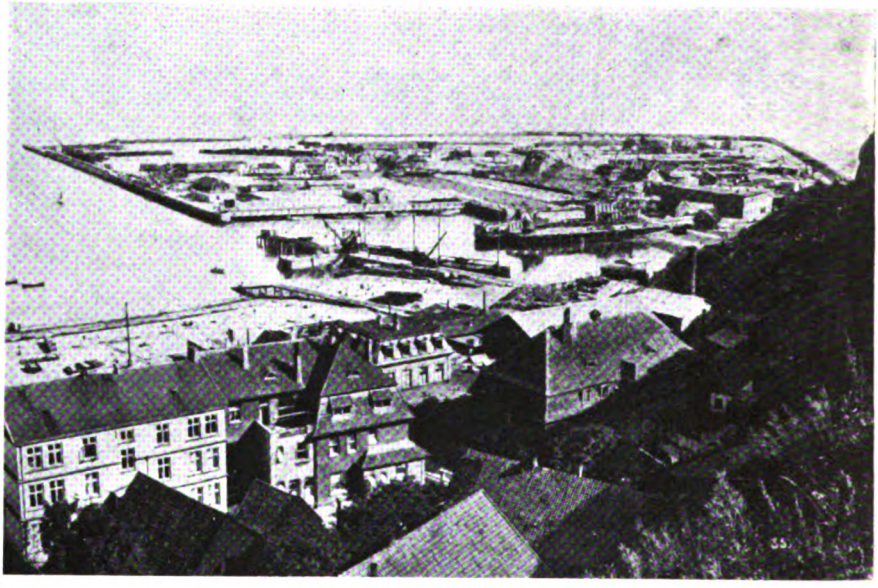

The Naval Harbour Complete.

The Harbour after Demolition.

Demolition of Heligoland Naval Base. (*See p.* 141.)

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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume XX. No. 3 LONDON, MARCH, 1925.

EDITORIAL NOTES.

Concrete Housing in the Netherlands.

AN acute shortage of houses coupled with a dearth of skilled building labour and high prices of building materials form a set of conditions from which this country is not alone in suffering. It is a condition common to every European nation, each of which is endeavouring in its own way to provide sufficient accommodation to house its inhabitants in comfort. All over the Continent there is thus not only an earnest attempt to build houses at a reasonable cost, but there is also an unprecedented concentration of attention on the problem of cheaper building. A study of what has been done abroad shows that we in this country have been no less sincere in approaching the problem, or spent less on housing, but there is this noticeable difference between our efforts and those of most of the continental countries: while in this country the problem has been looked upon as one of building houses of a better standard and with more amenities than the working classes were accustomed to before the war, as a rule our neighbours have considered the problem to be the erection of as many temporary dwellings as possible for a limited outlay, so that the bulk of the money available for housing may be reserved for the erection of permanent buildings when costs reach a more reasonable level. Thus we find houses of wood and corrugated iron being built in considerable numbers, and in Austria the "Kernal house," consisting of two or three rooms only which may be added to as funds permit, is favoured.

In Holland, however, as in this country, the permanent house is preferred, and efforts have been concentrated on the erection of sound and substantial structures at the lowest cost. In the Netherlands the brick is the traditional building material, but when it was found that brick houses at over 200 per cent. more than the pre-war cost were prohibitive, whole-hearted efforts were made to find a substitute which would give an equally permanent structure at a price which would allow houses to be built in sufficient numbers. The city of Amsterdam led the way, and Dr. Kepler, Director of Housing at Amsterdam, investigated no less than forty different building systems. From these he selected ten, some of which had already been successfully used for a large number of houses and are now being used for the construction of a garden city of a thousand houses. The success of the adoption of these alternative methods, all of which are concrete, on a large scale may be gathered from the fact that from the level of 225 per cent. over pre-war prices ruling in 1921 the cost of building working-class houses has now fallen to only 50 per cent. above the 1914 level. The methods adopted for

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by far the majority of the new houses in Holland make use of concrete poured *in situ* or large slabs, in some cases the whole side of a house being cast in one piece. The ideas underlying these systems are not new to us, for Mr. John Brodie, the City Engineer of Liverpool, many years ago built houses of large concrete slabs, and the late James Watson, F.R.I.B.A., experimented with casting whole walls horizontally and erecting them by crane, but so far the methods used in Amsterdam have not been extensively used here. The particulars and illustrations in this issue, for which we are indebted to the courtesy of Dr. Kepler, form the first detailed account of the systems used in Holland which has been published in this country. The second interim report of the Committee on New Methods of House Building, which dealt with concrete and recommended that more attention be given to *in situ* and large slab construction, was obviously influenced by what its members had seen in Holland, and the article in this issue should be of interest to all concerned with housing.

Formwork.

PERHAPS because it is an accessory to the erection of concrete structures, and does not form part of the finished work, the subject of shuttering has received but little attention. In many cases it is simply a matter of the foreman guessing the sizes of timber required to support the concrete and the carpenter cutting the material to fit, with the result that often more timber is used than is necessary and a great deal is cut to waste. In the building industry the cost of tools and plant accounts for a bigger proportion of the working expenses than in most other industries, and therefore presents a larger field for leakage or economy, and with the extending use of concrete for all kinds of building and constructional work the item of shuttering will become of increasing importance. We have frequently pointed out the advantage of designing formwork in the drawing office rather than leaving it to " work itself out " on the job, and the series of articles we are now publishing emphasises the importance not only of the formwork being designed in the office but also of the necessity of keeping a thorough check on the material on the site to ensure that it is not wasted.

The competition announced by the Ministry of Health (the conditions of which are given elsewhere in this issue), however, opens up another aspect of the problem of shuttering, namely, the design of shuttering for the construction of small houses. Following the recommendation of the Committee on New Methods of House Construction that concrete cottages should be extensively built by local authorities, and that monolithic walling should be adopted to a greater extent in order to obviate the necessity of employing bricklayers, this question becomes one of much importance. In the conditions of the competition the Ministry of Health clearly lay down the essential qualities of the ideal shuttering for small houses ; it must be economical, it must leave the wall faces so that they can be finished with a minimum of further treatment, it must be durable, it must be adaptable for use with different designs, it must be possible to fix and refix it by unskilled labour, and it must be adaptable for the construction of cavity walls. These qualities are all obviously required, but it will not be easy to combine them in one system which is economical and does not require the use of a large number of units of different sizes. We agree with the Minister of Health that the last word has not yet been said on the subject—the same can be said

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of all processes and materials used in building construction—but the patentees of the systems already on the market have spent a great deal of thought and shown much ingenuity in the methods with which we are already familiar, and it remains to be seen whether anything better can be devised during the few months of the competition. At any rate the prize is a good one, which should tempt all those with ideas on the subject to put them on paper.

Roads of the Future.

THE objects of the new Roads Improvement Bill which is shortly to be introduced in the House will be welcomed by all, for the enormous cost of building and maintaining roads to carry present-day traffic affects every ratepayer in the country. Although roads are under the care of local governing bodies, the question is a national one, for history shows that one of the vital factors in the prosperity of a country is the ease with which men and goods can move or be moved from one part to another, and with the increasing use of road transport of goods by heavy and fast vehicles the provision of the most suitable type of road is more than ever important. The Roads Improvement Bill is being specially introduced to confer powers on the Ministry of Transport to carry out experiments and trials on a national scale on road bases and surfaces with a view to ascertaining the methods and materials which will give the longest life with the lowest maintenance costs, and which at the same time will not be prohibitive in first cost. It is not proposed to confine the tests on the experimental lengths of road to a question of wear and tear under ordinary traffic conditions. Special tests will, we understand, be made as to the slipperiness of various kinds of road surfaces, a subject of great importance in view of the fact that a bad skid is almost the greatest danger the motorist has to face. There will be tests, too, of the effect of particular types of vehicles, such as lorries and omnibuses, in the deformation of the road surface and in setting up corrugations. At certain points along the test surfaces it is likely that slight ridges will be placed in order to find the effect of the impact after a slight drop of different types and sizes of tyres, such as steel, solid rubber, and pneumatic, and how they start deformation and corrugation. Considerable attention will be paid to the springing of vehicles, which is believed now to be much more important than has been recognised in the past. On this question *The Times* states that in some quarters it is believed that as much of the deformation and corrugation of the road surface is due to bad springing as to anything else. The springs of a heavy lorry are usually built to be most effective when it is loaded. If the load is 12½ tons, 8½ tons on the back axle and four in front, the springs are suitable to the load. The same lorry unloaded may be a practically rigid vehicle, and as when empty it frequently travels at a high rate of speed its capacity for damage may be considerable. Experiments have proved that with some lorries travelling at a high rate of speed the wheels are not on the road all the time, but there is a series of impacts which may quite easily start movement of the surface of the road. On a hot day, with a softened road surface, a very small movement may quickly become a big wave and break the surface.

If the Bill be passed into law these experiments will be watched with the greatest interest, and should go a long way towards settling some of the contro-

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versial statements so often seen as to the merits of different materials. As might have been expected, considerable opposition to the extended use of the all-concrete road is being shown by those interested in older methods and materials, but on a question involving an expenditure of over forty million pounds a year no trade interests should be allowed to stand in the way of the adoption of better or cheaper materials. We gather that one of the principal objects of the Ministry will be to produce a surface which will prevent skidding, and in this direction the concrete surface will, we think, be difficult to improve upon. In spite of the statement recently made in the press that concrete surfaces "dust up" and form a film of slippery mud, we prefer to draw our opinion from experience, which shows that it is practically impossible to skid a car on an all-concrete road under no matter what weather conditions.

Heligoland.

ONCE more in its chequered career the island of Heligoland makes a call on the historian. Formerly a Danish possession, it came into the hands of the British in 1807, and owing to its proximity to their coast was envied by the Germans until, amidst much misgiving in this country, it was handed over to Germany in 1890 in exchange for Zanzibar. From the time it came into their possession until the Armistice nothing was spared in the endeavour to make it impregnable; the island formed the outer defence to the Kiel Canal, and in twenty-four years no less than £35,000,000 was spent in harbour works and fortifications on its small area measuring one mile long by one-third of a mile wide. The island rises almost vertically out of the sea to an average elevation of nearly 170 ft. above high-water, with the fortifications hewn out of the solid rock, and it has now been the scene of what is by far the largest work of deliberate demolition ever undertaken. An idea of the enormous masses of reinforced concrete used for protecting the guns and underground chambers may be gathered from the photographs and notes on the work which we are able to give in this issue by the courtesy of the Institution of Civil Engineers, and Mr. L. H. Savile, C.B., M.Inst.C.E., the Civil Engineer-in-Chief to the Admiralty, under whose direction the work was carried out. The fruits of twenty-four years of patient toil were reduced to impotency in the space of a little over two years, and the island is now gradually returning to its former peaceful pursuits.

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House Building Methods in Holland.

Concrete Houses Built of Large Slabs.

At the cessation of hostilities Holland, although neutral, found herself faced with a shortage of houses as acute as that experienced by any of the belligerent nations. The cost of materials and wages rose by well over 200 per cent. between 1914 and 1921, but largely owing to the use of methods which do not require bricks and other materials which formerly formed the staple building materials, or any considerable amount of skilled labour, it was possible to reduce building costs in 1924 to a level of about 50 per cent. above the 1914 level.

The new material which has brought about this reduction is concrete, and many new systems of construction are being used with success. Steel houses do not appear to have received any consideration at all in Holland. Dr. Kepler, the Director of Housing at Amsterdam, to whom we are indebted for most of the information in this article and for kindly lending us some of the accompanying illustrations, has inspected demonstration steel houses in this country, but is apparently not impressed with them; perhaps it was for fear of wounding our susceptibilities that he declined to give an opinion on them, and contented himself with telling us that steel houses were "very interesting!"

It is at a garden city now being built for the Amsterdam Municipality that various systems of concrete construction are now being extensively used. As a preliminary step no less than forty building systems were tested, as a result of which ten different systems were selected for the erection of 900 concrete houses; thus an average of 90 houses are being built on each system, so that there is an excellent opportunity of comparing the speed with which they can be erected on fairly large schemes. The ten different methods in use are as follows:

(1) *The "Kossel."*—Walls poured *in situ* between wooden shuttering. Concrete composed of Portland cement, slag from blast furnaces, and clinker from refuse destructors.

(2) The "*Greve.*"—Walls poured *in situ* between shuttering made up of wooden frames in which sheets of iron are fixed. Concrete made of Portland cement and slag passing a ¾-in. ring and retained on a ⅜-in. sieve.

(3) *The "Non-Plus."*—*In situ* concrete, formed of Portland cement, slag, and sand, poured between standardised wooden shuttering.

(4) *The "Occident."*—Whole walls poured in moulds on the site and erected by crane.

(5) *The "Bron."*—Pre-cast slabs, about 100 sq. ft. in area, pre-cast and erected by crane. Each slab is the height of a story, and contains openings for door and window frames. The slabs are strengthened by reinforced concrete ribs.

(6) *The "Hunkemoller."*—Pre-cast slabs one story high by 1 ft. 8 in. wide, erected by crane.

(7) *The "Isotherme."*—Concrete block cavity wall. Outer and inner leaves each 3½ in. thick; cavity 4 in. Blocks measure 11 in. long by 8½ in. high by 3½ in. thick. Blocks for outer leaf made of gravel concrete and the blocks for the inner leaf of slag concrete; the blocks for both leaves are hollow.

Door Frame being lifted into Position.

The "Occident" System.

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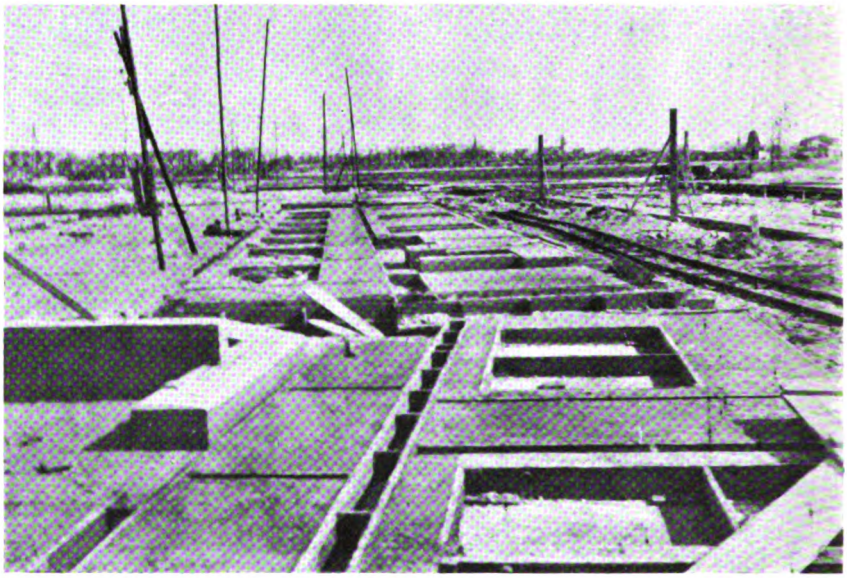
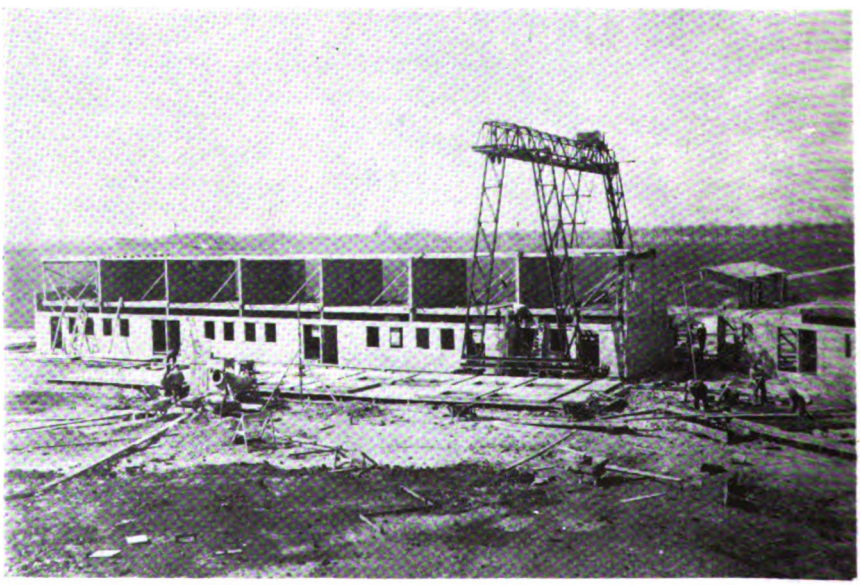


Wall Slabs Maturing.



Walling of Ground Story in Place: First-Floor Wall Slabs being lifted into Position.

Housing in Holland: The "Occident" System.

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Walling nearing Completion.

Walling Completed.

Housing in Holland : The " Occident " System.

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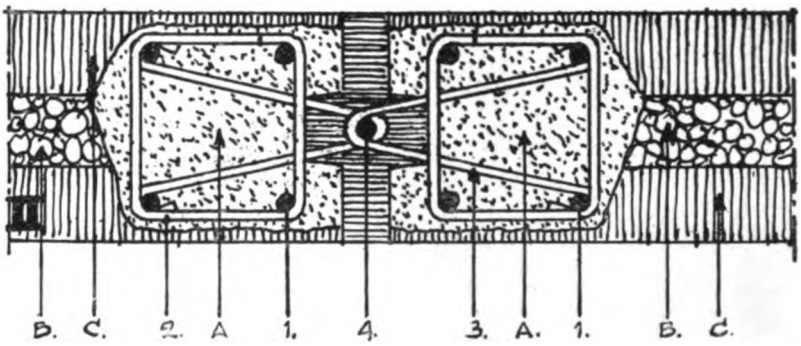

A = Reinforced Piers ; B = Filling of Clinker without Cement ; C = Inner and Outer Faces of Breeze Concrete. Note : For external walls impervious concrete is used for one of the Faces (C).

The " Occident " System : Showing Connection of Slabs.

(8) *The " Winget."*—This system of concrete block construction is well known in this country. It is composed of blocks measuring 18 in. by 9 in. by 4 in., with a cavity of about 2½ in., the outer leaf being of impervious concrete and the inner leaf of breeze.

(9) *The " Olbertz."*—Hollow impervious concrete blocks, 2 ft. 6 in. by 8 in. by 6½ in., with ribs on the inner face against which is fixed a sheet of pervious concrete.

(10) *The " Korrelbeton."*—Porous concrete poured *in situ* between metal shuttering in wooden frames. This system was fully described in our issue for October, 1923.

In addition the " Binns " system of pouring concrete between thin sheets of concrete, which act as formwork and are left in position as the face of the wall, and the " Dorlonco " system of steel-frame and concrete construction are also being used for a few houses.

All the houses are two stories in height. Dr. Kepler tells us that, chiefly owing to the necessity of employing bricklayers, he has used the concrete block methods for a few houses only, and that the majority are being erected by *in situ* methods or by the use of large slabs, such as the " Occident," the " Hunkemoller," the " Korrelbeton " and the " Kessel " systems. By these methods a minimum of skilled labour is required, and the houses can be erected by unemployed labour from other trades providing sufficient skilled supervision is provided. Owing to the opposition of the trade unions to other than building trade labour being employed, the houses are now being erected by building trade labourers, and the average cost is working out at £350 per house, including the pile foundations necessary owing to the water-logged condition of the ground.

As will be seen from the list above, the systems may be divided into three classes : (1) concrete poured *in situ* between shuttering ; (2) large slabs erected by crane ; and (3) concrete blocks. The use of concrete blocks has been thoroughly explored in this country, and the chief interest lies in the methods of using large pre-cast slabs ; two typical systems of this type of construction and a method of monolithic wall construction are here described.

The " Occident " System.

This method of construction consists of pre-cast reinforced concrete walls made on a platform on the building site. Each of the outer and partition walls is poured complete on a platform in a horizontal position, with openings for doors and windows formed by timber falsework. The floors are also pre-cast. The walls have a total thickness of 6 in. and are composed of porous concrete for the inner face, a core of loose clinker, and an outer impervious face, as shown in the diagram on this page. The porous inner face of the wall, consisting of 2 in. of clinker concrete in which is embedded a mesh of reinforcement, is poured first, and on this is placed a layer of 2 in. of loose clinker without cement or other binding agent ; the

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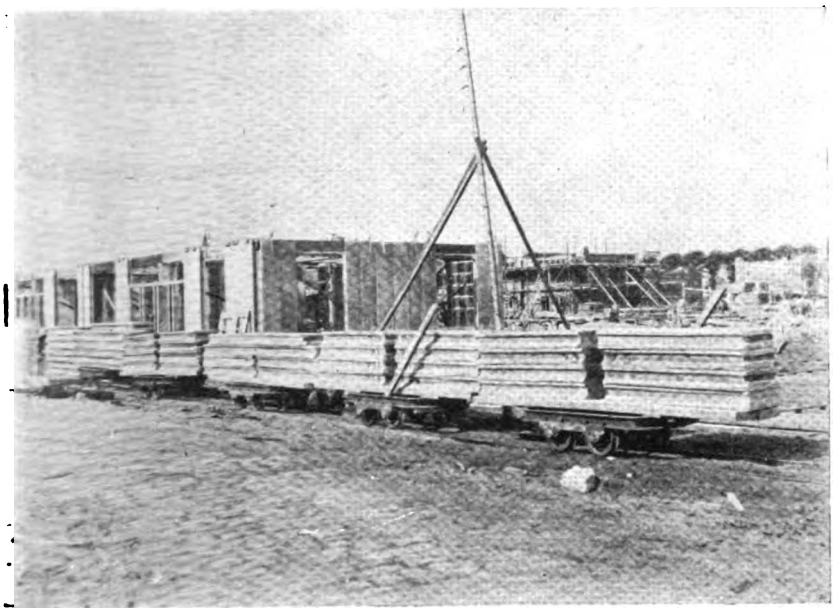
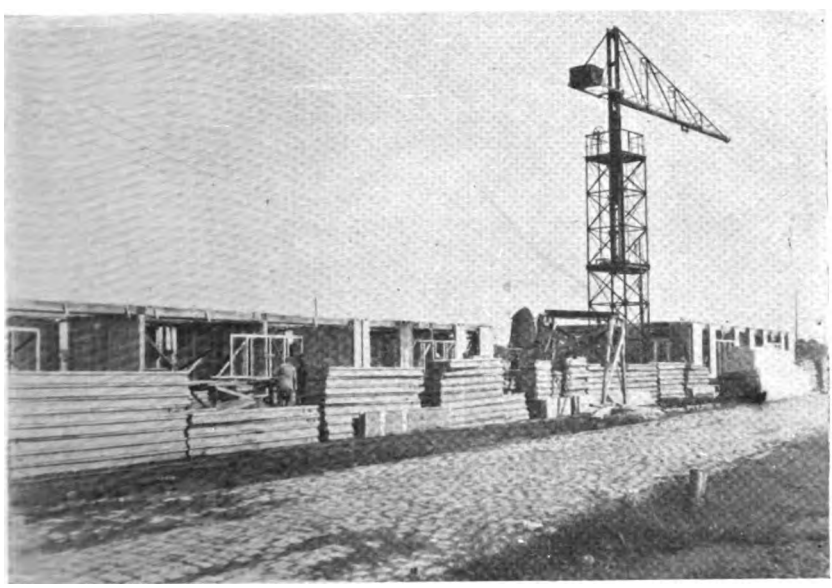


Transporting Slabs to Site.



Erecting Slabs by Crane.

Housing in Holland: The "Hunkemoller" System.

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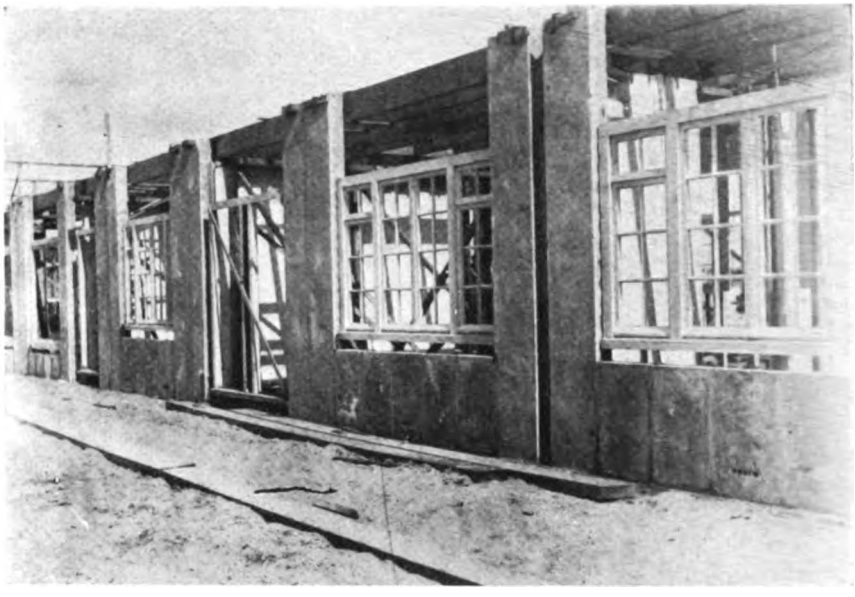

Showing Method of Fixing Window Frames.

Showing Method of Erecting First-Floor Slabs.

Housing in Holland: The "Hunkemoller" System.

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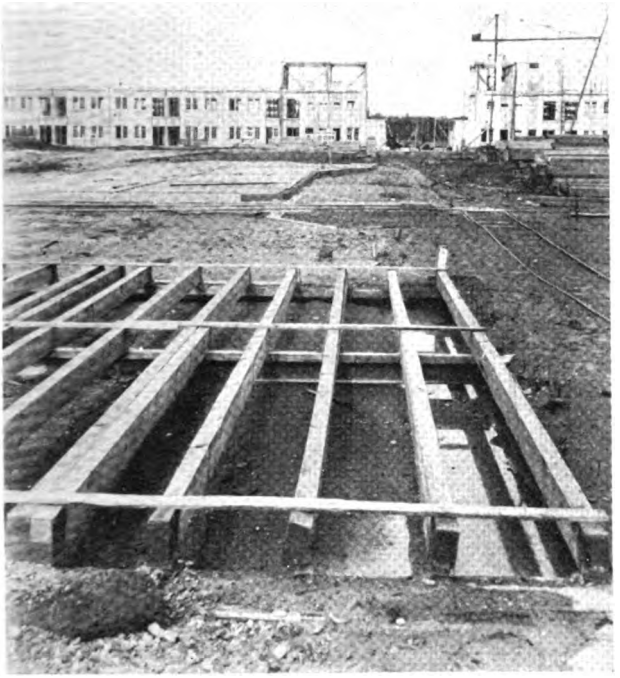

Houses nearing Completion in Background. Concrete Raft Foundation in Foreground.

Fixing Flooring at First-Floor Level.

Housing in Holland: The "Hunkemoller" System.

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General View of Site.

Showing Walling Completed before Roughcasting.

Housing in Holland: The "Hunkemoller" System.

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outer face of the wall is then laid in the form of 2 in. of 1 : 4 gravel concrete in which is embedded a second layer of mesh reinforcement. Four reinforcing rods, wired, are inserted at each side of the slab to form reinforced concrete columns when they are erected, the ties being so formed that they project from the edges of the slab. As soon as possible after pouring the surface of the slab is brushed in order to expose the aggregate and give an attractive surface finish, and after ten days the slabs are placed in position by a crane. The slabs measure approximately 220 ft. super, and weigh about 3 tons.

After they are placed on the prepared footings by the crane the slabs are temporarily held in position until the four walls have been erected and securely fixed at the corners by iron rods passing through the eyes formed by the projecting ties and concreted in.

To facilitate handling by crane, iron rings are cast into the top of the walls and afterwards removed. Door and window frames are placed in position and fixed with cement.

The crane used is of the " transporter " type, mounted on narrow-gauge tracks at each side of the row of houses to be built ; the system is thus most economical on long rows of houses, and while on a congested site there may be difficulty in following a typical English garden city lay-out with houses set at different angles and set back varying distances from the road, this could probably be overcome by collaboration between the architect and the builder.

The "Hunkemoller" System.

In this system slabs generally the full height of the story by 1 ft. 8 in. wide are made in a factory near the site and transported to where they are required on a narrow-gauge railway. The slabs are made in wooden frames, and are composed of 1 : 5 concrete for the inner and outer faces with a core of gypsum and porous aggregate in the centre; reinforcing rods are inserted in the slabs as they are made. These slabs are used for the inner as well as the outer leaves of the walls ; when used for internal walls they are finished with plaster, which is applied and screeded whilst the slabs are in a horizontal position and gives a surface smooth enough to enable plastering to be dispensed with after they are erected, except to make good the joints. The slabs are formed with a > joint on each edge, and when they are in position the joints are plastered with a trowel. This plastering serves the double purpose of making good the joints and acting as shuttering when concrete is poured from the top down the < > shape cavity to bind the slabs together. As is shown in the illustrations, these slabs are also erected by the crane.

The size of the openings for the doors and windows is a multiple of the width of the slab, and the window and door frames are so shaped at their edges that they may be slotted from the top into the < > grooves in the slabs. Above and below such openings slabs of a shorter length are used. The external faces of the walls are roughcast or rendered.

In both these systems the roofs may be either flat, pre-cast and hoisted by crane, or pitched and covered with tiles or slates.

In Holland, and particularly by Dr. Kepler, a very close study has been made of the problem of the rapid and cheap production of houses, and there is no doubt that their inspection of the methods illustrated herewith largely influenced Sir Ernest Moir and the members of the Committee on New Methods of House Building when they drew up the report given in our last issue, in which the use of large slabs was strongly recommended. The attention of the architectural profession has already been attracted to the work of the younger Dutch architects in the design of small houses, and certainly much can be learnt from their use of new materials. Dr. Kepler informs us that he would be pleased to welcome English architects who would like to see what is being done in Amsterdam, an invitation which we hope will be taken advantage of by all responsible for housing.

The "Korrelbeton" System.

The " Korrelbeton " system is already well-known in this country, and is to be used on a large scale by the Edinburgh Corporation, and experimental houses are shortly to be built in different parts of the country.

The method of construction is to pour a porous concrete between a special and economical type of shuttering consisting of a wooden framework filled in with metal plates, and placed in position

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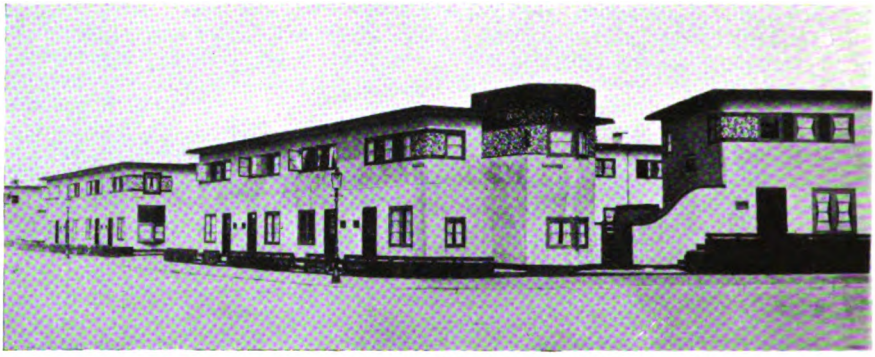

"Korrelbeton" Houses.

in sections of a height corresponding to that of each story of the building to be erected. The shuttering of each story can be removed irrespective of the shuttering of the other stories, and the plates can be removed from the framework whilst it is in position.

The concrete is made from furnace slag, broken brick, coke breeze, or other angular and porous aggregate to pass a ½-in. mesh, bound together with Portland cement without the addition of fine material to fill the voids. For walls the proportion of cement and aggregate is 1 : 10, while for floors and roofs, in which case a finer aggregate is used, the proportion is 1 : 5 or 1 : 3. The resultant concrete is exceedingly light and porous, and it is therefore possible to use light shuttering. The voids in "Korrelbeton" equal from 30 to 50 per cent. of the total volume of the concrete, and this, it is claimed, renders walls built of this material non-conducting of heat and cold, sound-proof, and fire-proof, while at the same time the light and porous concrete prevents condensation and permits of nails being driven into the walls. The outer walls are usually 12 in. thick for foundations and 8½ in. above ground level; divisional walls are usually 4 in. thick.

The method of erecting these houses is as follows. A raft foundation is first laid, consisting of a bottom course of 4 in. of "Korrelbeton" with the same size aggregate as is used for the walls, reinforced with a layer of mesh reinforcement laid 1 in. from the bottom, and a top course of 2 in. of "Korrelbeton" made with a finer aggregate containing only from 10 per cent. to 20 per cent. voids, thus forming a more dense and stronger concrete. The walls are then poured up to first-floor level, and the first floor laid on the same lines as the ground floor. The walls are then carried up to second-floor level and the second floor laid, and so on to any desired height.

It is claimed that the economies which are possible by this system, both in the cost of the concrete and the cost of erection, amount to about 20 per cent. as compared with brickwork. The shuttering is standardised in units which allow any architectural design to be followed, and we are informed that between 300 and 500 houses per year can be built with a plant costing £3,000.

The Director of the Municipal Building and Housing Council at The Hague, and other Dutch and French building authorities, have reported favourably on this system. Tests on 1 sq. yd. of "Korrelbeton" 6 in. thick, such as is used for floors, supported at each of the four corners, showed no deflection when loaded with 2,690 lb. in the centre of the slab.

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DEFLECTION OF OVERHANGING BEAMS.

By S. M. MILBOURNE.

THE object of this brief communication is to investigate from a mathematical standpoint the deflection of overhanging beams of constant cross section, under the influence of point and distributed loads. The bending moment and shearing force for beams of this class have been fully dealt with in most of the text-books; the subject of deflection, however, has received but little attention.

Figs. I, II, III and IV represent to an exaggerated scale the most characteristic forms of the elastic line for a beam simply supported at points distant (na) from each end and subjected to symmetrical concentrated or point loads W and W_1.

In considering the beam under the influence of the loading condition just described, it will readily be understood that should the central load W_1 be greatly in excess of the cantilever loads W, or the length of the centre span ($2a$) be much greater than the span length (na), the elastic line for the beam would take a form similar to that shown in *Fig.* I. In this case the resulting deflection of the centre span (δ_1) would be positive in sign, and the deflection of the cantilever ends (δ_2) would be negative in sign when measured from the support levels.

When the point loads on the cantilever ends W are each greatly in excess of the central load W_1, or the span length (na) is much greater than the span length ($2a$), the elastic line of the beam will assume a form similar to that shown in *Fig.* II. In this case the deflection of the centre span (δ_1) would be negative in sign, and the deflection of the cantilever ends (δ_2) would be positive in sign.

The cases just cited represent the formation of the elastic line for two extreme conditions of loading, and it will be seen from *Figs.* I and II that maximum values of positive and negative deflection obtain at the centre of the beams and at the ends of the cantilevers in each case.

Figs. III and IV represent the formation of the elastic line for intermediate conditions of loading and span lengths.

In *Fig.* III slight local deflection has taken place under the point load W_1 at centre of beam, but this load is not of sufficient magnitude to overcome the deflecting influence due to the cantilever loads W, W, thus resulting in a negative slope over the supports and a maximum negative deflection at points P, P on the middle span.

In *Fig.* IV the point load W_1 is of such magnitude as completely to overcome the deflecting influence due to the cantilever loads W, W, causing a positive slope over the supports. This slope, however, is not sufficient to prevent dipping due to the deflection of the cantilever ends, and consequently the maximum value of negative deflection is reached at points P, P, along the cantilevers. It is therefore necessary in finding the true shape of the elastic curve to examine the loaded beam for these points with the object of obtaining their magnitude and position.

The above examples are given in order to demonstrate more clearly the possibilities of the elastic line under different conditions of loading. Variations in the span lengths are, of course, quite as effective as load variation in influencing the deflection.

What has been stated with reference to the action of point loads is equally

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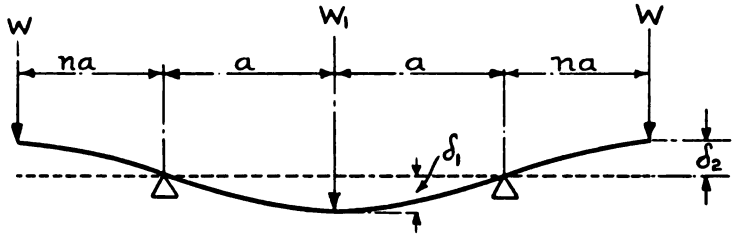

FIG. I.

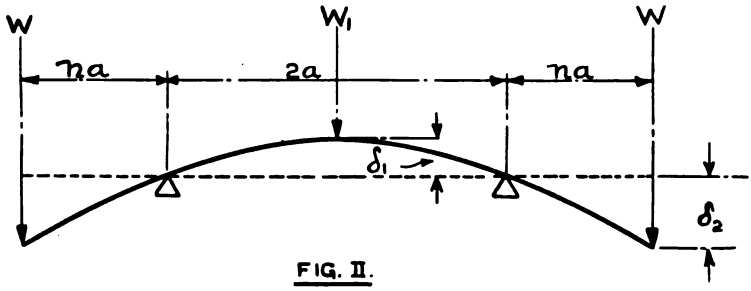

FIG. II.

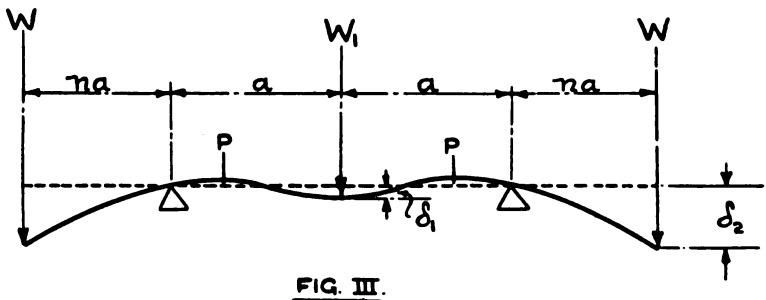

FIG. III.

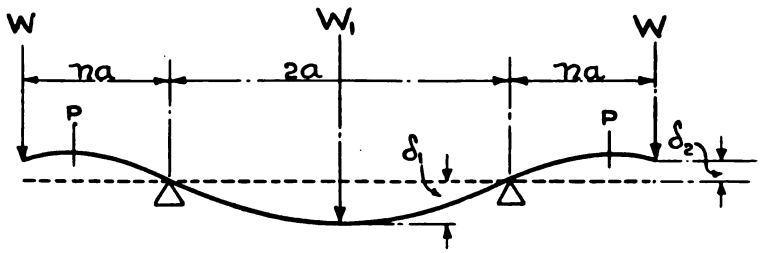

FIG. IV.

DEFLECTION OF OVERHANGING BEAMS.

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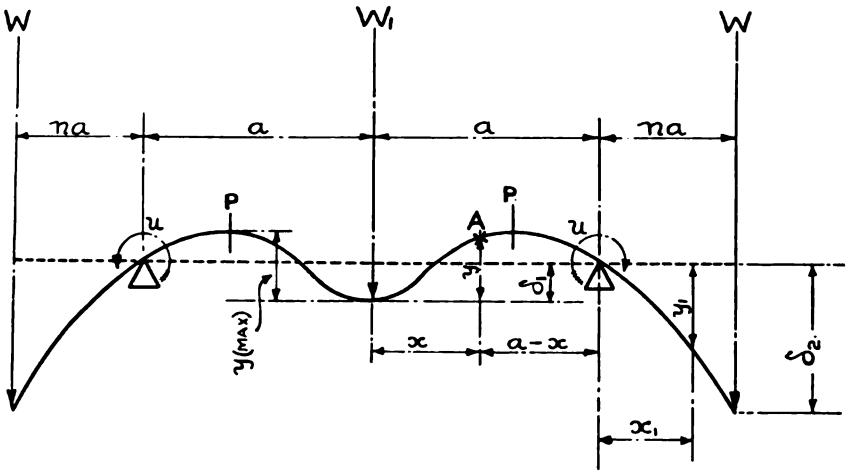

FIG. V

applicable to the condition of distributed loads. *Fig.* V represents the loading condition for investigation.

Now, considering any point A on the beam between the supports and a distance x from centre of beam, the bending moment will be $\frac{W_1}{2}(a - x) - u$ in. tons, where (u) equals the counter moments at the supports due to the loads (W) and the cantilever lengths (na).

Then, substituting values for (u) in the bending moment formula and applying the usual mathematical relation to determine the deflection (y) at point (A) assuming the beam is of constant cross-section, the following is obtained :—

$$EI\frac{d^2y}{dx^2} = \frac{W_1}{2}(a - x) - Wna$$

and on integrating once :

$$EI\frac{dy}{dx} = \frac{W_1}{2}\left(ax - \frac{x^2}{2}\right) - Wnax + C,$$

but at centre of beam where $x = 0$, $\frac{dy}{dx} = 0$, since the loading is symmetrical,

$\therefore$ constant $= 0$.

$$\therefore EI\frac{dy}{dx} = \frac{W_1}{2}\left(ax - \frac{x^2}{2}\right) - Wnax. \quad . \quad . \quad . \quad . \quad . \quad (1)$$

This gives the slope of the beam at any point A, distant x, from the centre of the middle span.

Therefore at the end of the middle span where $x = a$ the slope will be :

$$\frac{dy}{dx} = \frac{W_1}{2EI}\left(a^2 - \frac{a^2}{2}\right) - \frac{Wna^2}{EI}$$

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which reduces to:

$$\frac{dy}{dx} = \frac{W_1a^2 - 4Wna^2}{4EI} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Then integrating equation (1)

$$EIy = \frac{W_1}{2}\left(\frac{ax^2}{2} - \frac{x^3}{6}\right) - \frac{Wnax^2}{2} + C,$$

but at the centre of beam where $x = 0$, $y = 0$. Therefore the constant $= 0$.

$$\therefore y = \frac{W_1}{12EI}(3ax^2 - x^3) - \frac{Wnax^2}{2EI} \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

and where $x = a$ in the above equation, $y = \delta_1$. Where δ_1 = deflection of centre of beam relative to the points of support

Then

$$\delta_1 = \frac{W_1}{12EI}(3a^3 - a^3) - \frac{Wna^3}{2EI}$$

which reduces to

$$\delta_1 = \frac{W_1a^3 - 3Wna^3}{6EI} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Referring to *Fig.* V, it will be seen that the maximum value of y on the centre span, occurring at points P, P, may not necessarily obtain over the points of support where $x = a$, but anywhere on the beam between these supports.

Now, remembering that wherever the point P, P, occurs the slope at that point must be zero, it follows that the distance x from the centre of beam to the point P between the supports may be found by writing $\frac{dy}{dx} = 0$.

or

$$\frac{W_1}{2EI}\left(ax - \frac{x^2}{2}\right) - \frac{Wnax}{EI} = 0.$$

Then, solving for the real value of x in the above equation the distance at which the maximum value of y occurs may be obtained, and by substituting the value of x in equation (3), the maximum value of y may be determined.

The deflection of the cantilever ends may be found from the formula giving deflection for a simple cantilever and adding the slope occurring at the supports (algebraically).

Let δ_2 = deflection of overhanging end.

Then $\delta_2 = \frac{W(na)^3}{3EI} + na\frac{dy}{dx}$

Then, substituting the value given by equation (2) in the above, the following is arrived at:—

$$\delta_2 = \frac{W(na)^3}{3EI} - \frac{na}{4EI}(W_1a^2 - 4Wna^2) \quad . \quad . \quad . \quad . \quad . \quad (5)$$

It will be observed that in equation (5) $nady/dx$ becomes a negative value when applied to the deflection formula for the cantilever ends. The reason is

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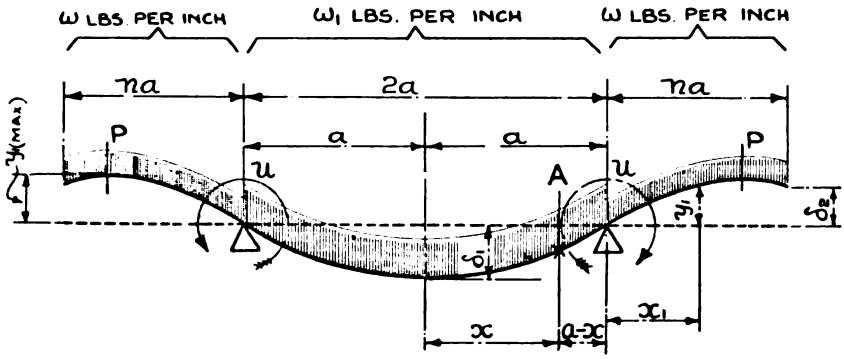

FIG. VI.

obvious since the slope of the elastic line on one side of the supports is downwards and must therefore be upwards on the other side.

Should it be found that no maximum value of y exists on the middle span of the beam between the points of support, the cantilevers may then be examined for a maximum value y_1 (as P, P, *Fig.* IV) in the following manner :

The formula giving the slope at any point distant x_1 along a simple cantilever under the influence of a point load is given by the equation

$$\frac{dy_1}{dx_1} = \frac{W}{EI}\left(nax_1 - \frac{x_1^2}{2}\right).$$

Therefore adding (algebraically) the initial slope at the point of support to the above expression, the slope of the overhanging portion of the beam may be found or any distance x_1 along the span.

Then

$$\frac{dy_1}{dx_1} = \frac{W}{EI}\left(nax_1 - \frac{x_1^2}{2}\right) - \left(\frac{W_1a^2 - 4Wna^2}{4EI}\right)$$

where $\frac{dy_1}{dx_1}$ = the actual slope of the cantilever ends any distance x_1 along the pan.

Then writing $\frac{dy_1}{dx_1} = 0$

or

$$\frac{4Wnax_1 - 2Wx_1^2 - W_1a^2 + 4Wna^2}{4EI} = 0,$$

and solving this equation for x_1 the distance from the point of support to the maximum value of upward deflection is obtained, and substituting its value in the equation

$$y_1 = \frac{W}{6EI}(3nax_1^2 - x_1^3) - \frac{x_1}{4EI}(W_1a^2 - 4Wna^2) \quad . \quad . \quad . \quad (6)$$

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the maximum value y_1 is obtained, where $\frac{W}{6EI}(3nax_1^2 - x_1^3)$ is the formula giving the deflection at any point x_1 along a simple cantilever beam with point load at end of span.

It should be noted that y_1 and δ_2 are measured upwards or downwards from the support levels according to the sign, positive sign denoting downward deflection and negative sign denoting upward deflection relative to the supports. If $W_1 = W$ the simplification of the above formula is obvious.

The deflection due to the influence of distributed loading will now be considered.

Fig. VI represents the loading condition for investigation where w_1 equals load per inch distributed on centre span and w equals load per inch on overhanging ends.

Now, considering any point A on the beam between the supports at distance x from the centre, then the bending moment will be $\frac{w_1}{2}(a^2-x^2)-u$ inch tons where u = counter moments at the supports and equals $w(na)^2/2$.

Then

$$EI\frac{d^2y}{dx^2} = \frac{w_1}{2}(a^2 - x^2) - \frac{w(na)^2}{2}$$

and integrating

$$EI\frac{dy}{dx} = \frac{w_1}{2}\left(a^2x - \frac{x^3}{3}\right) - \frac{w(na)^2x}{2} + C$$

but at centre of beam where $x = 0$, $\frac{dy}{dx} = 0$, $\therefore$ constant $= 0$

$$\therefore\ EI\frac{dy}{dx} = \frac{w_1}{2}\left(a^2x - \frac{x^3}{3}\right) - \frac{w(na)^2x}{2}. \quad . \quad . \quad . \quad . \quad . \quad (1a)$$

The above equation gives the slope of the beam at any point A distant x from the centre of the beam.

Therefore at the end of the centre span where $x = a$ the slope will be

$$\frac{dy}{dx} = \frac{w_1}{2EI}\left(a^3 - \frac{a^3}{3}\right) - \frac{w(na)^2a}{2EI}$$

which reduces to

$$\frac{dy}{dx} = \frac{2w_1a^3 - 3wn^2a^3}{6EI} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2a)$$

Then integrating equation (1*a*)

$$EIy = \frac{w_1}{2}\left(\frac{a^2x^2}{2} - \frac{x^4}{12}\right) - \frac{w(na)^2x^2}{4} + C,$$

but at centre of beam where $x = 0$, $y = 0$ therefore $C = 0$

$$\therefore\ y = \frac{w_1}{24EI}(6a^2x^2 - x^4) - \frac{w(na)^2x^2}{4EI}$$

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which reduces to

$$y = \frac{w_1(6a^2x^2 - x^4) - 6w(na)^2x^2}{24EI} \quad . \quad . \quad . \quad . \quad . \quad . \quad (3a)$$

and when $x = a$ in the above equation, $y = \delta_1$ where $\delta_1 =$ deflection of centre span relative to the points of support.

$$\therefore \delta_1 = \frac{w_1(6a^4 - a^4) - 6w(na)^2a^2}{24EI}$$

which reduces to

$$\delta_1 = \frac{5w_1a^4 - 6wn^2a^4}{24EI} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4a)$$

It has already been shown in *Fig.* V that the maximum value of y, occurring at points marked P, P, on the centre span may not necessarily occur over the supports where $x = a$, or at the centre of beam where $x = 0$. This also applies to the case of distributed loadings; therefore the distance x from centre of beam to the maximum value of y may be found similarly to the previous example, by writing $\frac{dy}{dx} = 0$, or

$$\frac{w_1}{2EI}\left(a^2x - \frac{x^3}{3}\right) - \frac{w(na)^2x}{2EI} = 0.$$

Solving for the real value of x in the above equation, the distance from the centre of beam to the maximum value y is obtained.

Then substituting the value of x thus found, in equation (3*a*) the magnitude of y may readily be determined.

The deflection of the overhanging ends may be found in a similar manner to the previous case for concentrated loads, but some modification of the formula is, of course, necessary.

The deflection of a simple cantilever subject to a distributed load is given by the formula $\frac{w(na)^4}{8EI}$. To this must be added (algebraically) the initial slope over the supports.

Let $\delta_2 =$ deflection of the overhanging end of beam, then

$$\delta_2 = \frac{w(na)^4}{8EI} + na\frac{dy_1}{dx_1}$$

and writing in the value of $\frac{dy_1}{dx_1}$

$$\delta_2 = \frac{w(na)^4}{8EI} - \frac{na}{6EI}(2w_1a^3 - 3wn^2a^3) \quad . \quad . \quad . \quad . \quad . \quad (5a)$$

Examining the cantilever ends for maximum value of y_1. The formula giving the slope of any point distance x_1 along a simple beam subject to a distributed load is

$$\frac{dy_1}{dx_1} = \frac{w}{2EI}\left\{(na)^2x_1 - nax_1^2 + \frac{x_1^3}{3}\right\}$$

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Adding the initial slope to the above expression, the actual slope of the overhanging portion of the beam may be ascertained for any distance x_1 along the span.

Then

$$\frac{dy_1}{dx_1} = \frac{w}{2EI}\left((na)^2x_1 - nax_1^2 + \frac{x_1^3}{3}\right) - \left(\frac{2w_1a^3 - 3wn^2a^3}{6EI}\right)$$

where $\frac{dy_1}{dx_1}$ = actual slope of the beam.

Then writing $\frac{dy_1}{dx_1} = 0$,

or

$$\frac{3wn^2a^2x_1 - 3wnax_1^2 + wx_1^3 - 2w_1a^3 + 3wn^2a^3}{6EI} = 0.$$

Solving the above for the real value of x_1 the distance to the maximum value of upward deflection is obtained; therefore substituting the value of x_1 in the expression

$$\frac{w}{2EI}\left(\frac{n^2a^2x_1^2}{2} - \frac{nax_1^3}{3} + \frac{x_1^4}{12}\right) - \frac{x_1}{6EI}(2w_1a^3 - 3wn^2a^3) \quad . \quad . \quad . \quad (6a)$$

the maximum value y_1 is obtained, where

$$\frac{w}{2EI}\left(\frac{n^2a^2x_1^2}{2} - \frac{nax_1^3}{3} + \frac{x_1^4}{12}\right)$$

is the equation, giving the deflection of a simple cantilever beam distance x_1 from fixed end.

The sign preceding the deflection in each case denotes positive or negative deflection as in the previous example.

Should the load be uniformly distributed over the whole length of the beam the modification in the above formula is obvious.

The Institution of Structural Engineers.

THE Institution of Structural Engineers has decided to form Sectional Committees of the Science Committee, to include representatives of the general body of members. Sectional Committees have already been formed to deal with (*a*) concrete, and (*b*) steel. Mr. R. H. H. Stanger is Chairman of the Concrete Committee, and is assisted so far by Messrs. A. C. Davis, Arthur E. Evans, Oscar Faber, H. Gardiner Lloyd, H. E. Steinberg and J. T. Saunders. The Steel Committee has Mr. S. Bylander as Chairman, with the assistance of Messrs. E. S. Andrews, H. Kempton Dyson, E. Fiander Etchells, J. Mitchell Moncrieff, H. J. Collins, and W. A. Green. Further Committees to deal with timber (Chairman, Mr. H. D. Searles-Wood), and brickwork and masonry (Chairman, Mr. E. Fiander Etchells) are in process of formation. Amongst the programme of matters awaiting report are such questions as Adhesion of and Friction between Concrete and Steel; Reinforced Concrete Piles; Effect of Sewage upon Concrete; Effect of Oils and Fats upon Concrete; Effect of the presence of Sulphur and its Compounds in Aggregates; Grading of Aggregates; Data regarding Elastic Moduli of Concrete for Stresses within Breaking Limits; Microscopic Examination of Minerals; Prevention of Dust from Concrete Floors; Electrolytic action in Reinforced Concrete; Economic Spacing of Beams and Supports in Warehouses; Girderless Floors; Co-ordination of Standard Specification for Structural Steel; Co-ordination of Building Regulations for the whole Kingdom, etc.

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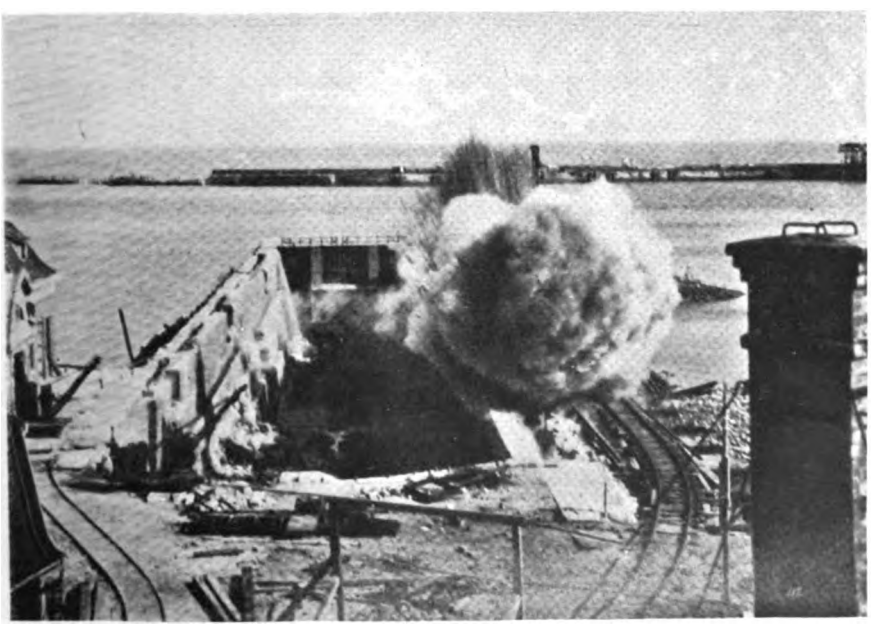
Demolition of the Defence Works at Heligoland.

THE demolition of the Harbour and defence works at Heligoland, which took twenty-four years to build and cost about £35,000,000, has now been completed under the direction of the Naval Inter-Allied Commission of Control. The actual demolition work was undertaken by the British Admiralty, whose senior representative on the Inter-Allied Commission was Admiral Sir Edward Charlton, K.C.B., K.C.M.G., and the bulk of the work was carried out under the direct supervision of Captain L.P. Willan, R.N., succeeded by Captain A. B. Cunningham, D.S.O., R.N., as President of the Sub-Commission ; Brig.-Gen. H. O. Knox, C.I.E., C.M.G., C.B.E., Commandant ; and Lieut.-Col. P. R. Warren, R.E., as Chief Engineer, representing Mr. L. H. Savile, C.B., M.Inst.C.E., Civil Engineer-in-Chief at the Admiralty.

An idea of the work entailed may be gathered from the facts that within a little more than two years no less than 195,676 cu. yds. of heavily reinforced concrete and brickwork were demolished, in addition to 53,416 cu. yds. of timber cribwork, a large number of caissons, and the whole of the guns and fortifications ; the work also included the depositing of concrete blocks in the entrance to the harbour so as to render it useless to any but small craft. The demolition of the 195,676 cu. yds. of concrete alone entailed the use of 295,611 lbs. of explosives, or 1·51 lbs. per cu. yd. During most of the time the work was in progress about 800 men were engaged.

For the following particulars of the work we are indebted to a paper read before the Institution of Civil Engineers on February 24 by Mr. L. H. Savile, who has kindly supplied us with the photographs illustrated in this issue.

The West Mole was by far the most important sea structure, and consisted of thirty-two caissons formed of steel plating on three sides, ends and bottom attached to an inner open steel skeleton framework. These caissons measured from 49 to 131 ft. in length to from 26 to 49 ft. in width, and were made



Demolition of Dry Dock.

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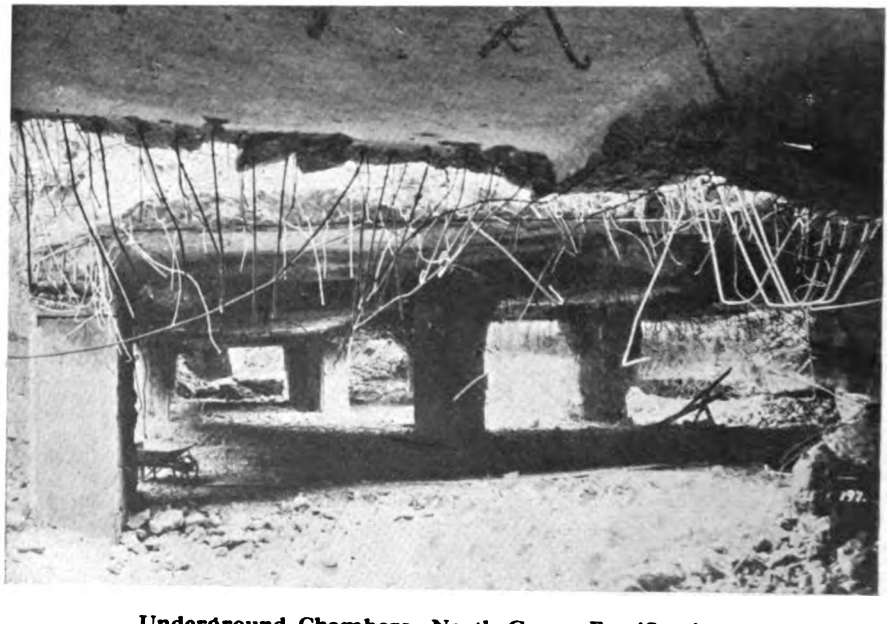
Underground Chambers, South Group Fortifications.



Underground Chambers, North Group Fortifications.

Demolition of Heligoland Naval Base.

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on shore, floated into position, and then filled with 1 : 5 concrete. A number of concrete blocks weighing 10 and 20 tons were placed outside the mole to protect the caissons. The whole of this mole was demolished. A typical caisson, with a total volume of 3,063 cu. yds., was demolished by boring forty-six holes to an average depth of 30·73 ft. and loading each boring with an average of 42·68 lbs. of explosives. Another caisson, with a volume of 1,745 cu. yds., was destroyed by drilling twenty-eight holes to an average depth of 22·64 ft. and loading with 96·25 lbs. of explosive. In the caissons with a width of 26 ft. the holes were spaced 7 ft. 1½ in. apart longitudinally and transversely in groups of four at 7 ft. 1½ in., 6 ft., and 7 ft. 1½ in. centres, the outer holes being 3 ft. from the faces. The cost of drilling 5-in. holes on the West Mole, with compressed air machines, worked out at an average of 0·65 men-hours per cu. yd.

The caissons on the North-East Mole were similar in construction to those on the West Mole. A total of 44,116 cu. yds. of concrete was destroyed, and 267 steel piles drawn. The labour cost of demolishing the superstructure was 3·36 men-hours per cu. yd., and for drilling caissons 3·17 men-hours per foot. A total of 24,787 cu. yds. of concrete was destroyed in the caisson portion of the North-East Mole, with the use of 27,712 lbs. of explosive; the effective drilling amounted to 7,582 ft.; the waste drilling to 2,339 ft.; the average depth of the 211 holes drilled was 35·932 ft.; the average charge per hole 131·34 lbs.; the explosive used per cu. yd. 1·118 lbs.; and the effective drilling per cu. yd. 0·306 ft.

The dry dock had a length of 367 ft. 8 in. from the inner groove to the head, with the pumping floor at level—6 ft. 3 in. The thickness of concrete at floor level, including rock stepped and left in, was 16 ft. 5 in., and at sea-bed level 13 ft. 6 in. A total of 14,300 cu. yds. of concrete was here demolished, with the use of 17,269 lbs. of explosive.

The main water tanks consisted of one large reinforced concrete tank divided into four compartments, two measuring 77 ft. 6 in. by 70 ft. 8 in. and the other two 77 ft. 6 in. square. The roof was supported on jack arches running in one direction, supported in turn on columns at 14 ft. 5 in. centres longitudinally and transversely. The depth of concrete in the floor and side walls was 9¾ in., while the columns had a cross-

Destruction of Observation Post and Howitzer Battery Emplacement.

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View Below Gunpit for 30·5 cm. Gun Turret "Dora" during Demolition.

section of 1 ft. 11½ in. by 1 ft. 3¾ in. The height from the floor to the underside of the roof was 9 ft. 11 in. The amount of concrete totalled 2,100 cu. yds., and cost in labour for drilling, handling explosives, and clearing débris 0·50 men-hours per cu. yd. The length of 1¾ in. diameter holes drilled was 217 ft., or 0·10 ft. per cu. yd., and the average weight of explosive used per cu. yd. of concrete was 2·56 lbs.

Three oil tanks were demolished, all 11 ft. 4¾ in. deep and 88 ft. 5 in. wide; two had a length of 88 ft. 5 in., and the middle tank 176 ft. 10 in. The concrete floor was 39·37 in. thick and the side walls 23·6 in. thick at the base and 11·8 in. at the top. The larger tank had a roof of steel plate covered with 2 in. of concrete and asphalte. In the middle tank the bottom was drilled with 2½ in. diameter holes 3 ft. 3 in. deep by Ingersoll-Rand tripod drills, and the side walls with 1½ in. holes from 9 in. to 18 in. deep. The total cost of drilling, maintaining tools, plant, handling explosives, cutting out and clearing débris in connection with the 1,126 cu. yds. of concrete in this tank was 3·34 men-hours per cu. yd.

The explosive used on the work is described as "Shell Filling C.88," which closely resembles "T.N.T."

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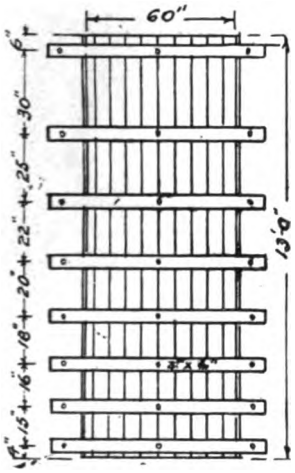
DESIGN OF FORMWORK FOR REINFORCED CONCRETE CONSTRUCTION.

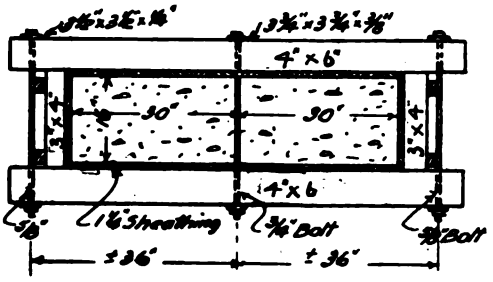
By A. E. WYNN, B.Sc., A.M.Am.Soc.C.E.

V—DESIGN PROBLEMS (Continued).

DESIGN 12: VERY LARGE COLUMNS.

Design forms for a column 16 *in. by* 60 *in.*, 13 *ft. high.*





DESIGN 12

To find what size yoke would be required without a centre support apply formula 37, assuming it is desired not to space the yokes closer than 9 in. If first yoke is 6 in. up, $h = 12{\cdot}5$, $1 = 5 + 1 = 6$ and $s = 9 = \dfrac{17{\cdot}06 \times bd^2}{12{\cdot}5(6^2 - 1)}$ from which $bd^2 = 231$.

If we use a yoke 4 in. wide, depth $= 7{\cdot}83$, or 4 in. by 8 in. is required. With this method of framing a width less than 4 in. would not be desirable.

For a deflection of $\frac{1}{8}$ in., formula 41, $d = \dfrac{32 \times 6^4}{6^2 - 1} = 11{\cdot}85$, so that deflection governs and a 4 in. by 12 in. must be used. This could be two 2 in. by 12 in., with bolt between. This, however, is too large practically, so a centre bolt will be used, and the spacing can be that for a 30 in. column.

From Table 9, using 4 in. by 6 in. yokes, placing first yoke 4 in. up, spacing will be 15 in., 16 in., 18 in., 20 in., etc., with $1\frac{1}{4}$ in. sheathing.

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Load on centre bolt at second yoke up

$$= 125 \times 11{\cdot}42 \times 1{\cdot}29 \times 2{\cdot}5 \times 2 \times \frac{1{\cdot}75}{3} = 5360 \text{ lbs.}$$

Diameter of threaded bolt $d = \sqrt{\dfrac{5360}{14000 \times 0{\cdot}785}} = 0{\cdot}70$, say, $\frac{3}{4}$ in. bolt.

Area of washer $= \dfrac{5360}{400} - 0{\cdot}785 \times 0{\cdot}875^2 = 14$, or side $= 3\frac{3}{4}$ in.

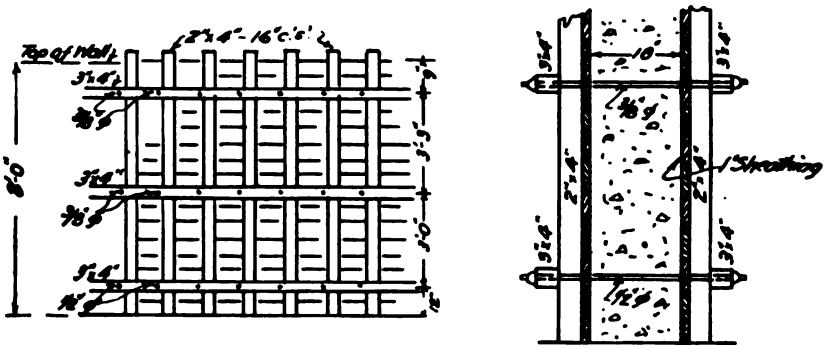
Thickness $= \dfrac{1}{220}\sqrt{\dfrac{400 \times 12{\cdot}82 \times 1{\cdot}3125}{1{\cdot}125}} = 0{\cdot}352$, say, $\frac{3}{8}$ in.

According to our rule the standard washer would be 4 in. by 4 in. by $\frac{3}{8}$ in. The two end bolts may be $\frac{5}{8}$ in. and the washers $3\frac{1}{2}$ in. by $3\frac{1}{2}$ in. by $\frac{1}{4}$ in.

The ends being only 16 in., smaller yokes may be used. We find from the tables that a 4 in. by 4 in. yoke will give approximately the same spacing for a 16 in. column as we are using for the 60 in. side, and therefore being end yokes we can use a 3 in. by 4 in. on its side.

DESIGN 13: LOW WALL FORMS.

Design forms for an 18 *in. wall,* 8 *ft. high.*



DESIGN 13

From Table 2, 1 in. sheathing will span 14 in., add 2 in. for stud, so place studs 16 in. on centre.

From Table 10, using 2 in. by 4 in. studs, the spacing of the ties or wales will be 12 in., 36 in., and 39 in.

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Load on lowest tie $= 7 \times 125 \times 1{\cdot}333 \times 2{\cdot}5 = 2915$ lbs.

Size of plain tie rod $d = \sqrt{\dfrac{2915}{20000 \times 0{\cdot}785}} = 0{\cdot}43$, or, say, $\frac{1}{2}$ in. diameter.

Load on second tie $= 4 \times 125 \times 1{\cdot}333 \times 3{\cdot}125 = 2085$ lbs., requiring a $\frac{3}{8}$ in. rod, and the same size would be used for the top wale or tie.

If the rods are to be drawn after stripping it would be better to use $\frac{1}{2}$ in. rods throughout to avoid having more than one size on the job.

Wire is commonly used for tying low wall forms, but the factor of safety is only about half what it is when bolts are used, since the wire is stressed up nearly to the elastic limit and to about one-half of its ultimate strength. Wires will seldom break from the pressure of the concrete, but they are liable to snap from sudden jars, as when hit by large stones or "plums" dropped into the wall, and they will cut into the stud or wale around which they are twisted, causing the form to give more or less.

Assuming a stress of 40000 lbs. per sq. in. in the wire, the area of wire required would be, for the lowest wale $= 0{\cdot}073$ sq. in., and the sectional area of No. 9 wire being 0·0163 sq. ins., four wires would be required, or two double strands. These strands are twisted tightly together inside the wall.

The same would be required for the centre wale and one double strand for the top wale.

Since with plain round rods it is assumed that patent clamps will be used, it is not necessary to use washers, as the clamp will give sufficient bearing.

DESIGN 14: HIGH WALL FORMS.

Design forms for a wall 18 ft. high, concreting to be completed in one operation.

From Table 2, $1\frac{1}{4}$ in. sheathing will span $13\frac{1}{2}$ in., using 3 in. by 4 in. studs; and, adding the thickness of the stud to the span of the sheathing, the studs will be 16 in. centre to centre.

From Table 10, spacing of the wales from the bottom will be 12 in., 36 in., 36 in., 39 in., 42 in. and 45 in., the last wale being 6 in. from the top of the wall.

					lbs.
Load from each stud on	1st	wale	$= 17 \times 100 \times 1{\cdot}33 \times 2{\cdot}5$		$= 5660$
,, ,, ,,	2nd	,,	$= 14 \times 100 \times 1{\cdot}33 \times 3$		$= 5600$
,, ,, ,,	3rd	,,	$= 11 \times 100 \times 1{\cdot}33 \times 3{\cdot}125$		$= 4600$
,, ,, ,,	4th	,,	$= 7{\cdot}75 \times 125 \times 1{\cdot}33 \times 3{\cdot}375$		$= 4400$
,, ,, ,,	5th	,,	$= 4{\cdot}25 \times 125 \times 1{\cdot}33 \times 3{\cdot}625$		$= 2600$

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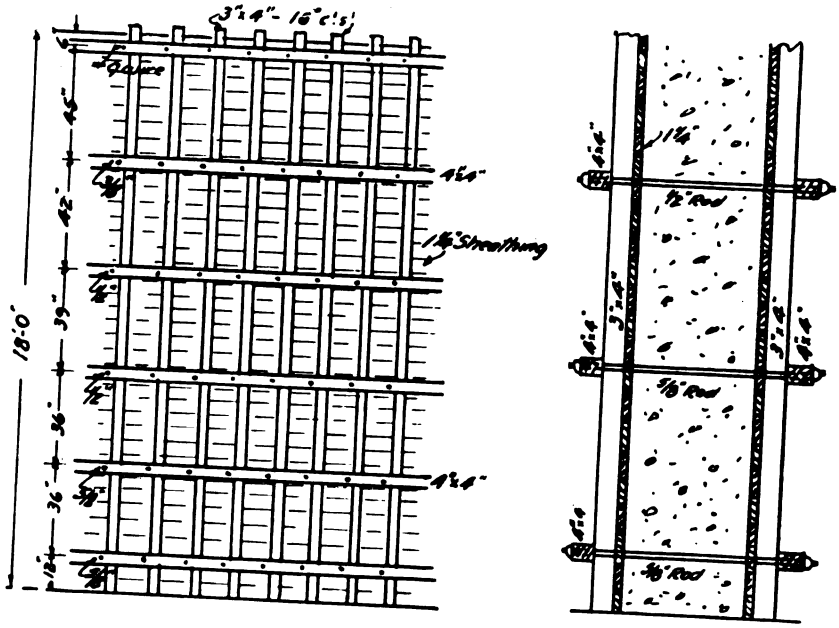
The exact loads by formulæ 52 and 53 will be successively 5530, 5600, 4545, 4305, 2515, and for the top wale 565 lbs. It will be noticed that these are very close to the loads found by the approximate method.



DESIGN 14

To find what size wale will be required if only alternate studs are bolted, $M = \frac{5600 \times 2{\cdot}66 \times 12 \times 8}{4 \times 10} = M_r = \frac{1600 \times bd^2}{6}$, or $bd^2 = 134$, or a 4 in. by 6 in. is required.

Load on the tie $= 5600 \times 2 = 11200$ lbs., and for threaded bolts $d = \sqrt{\frac{11200}{14000 \times {\cdot}785}} = 1$ in. diameter, or with plain rods $d = \sqrt{\frac{11200}{20000 \times {\cdot}785}} = 0{\cdot}84$ in., or, say, $\frac{7}{8}$ in. round rod.

With 1 in. bolt, area of washer $= \frac{11200}{400} + 0{\cdot}785 \times 1{\cdot}125^2 = 29$ sq. in., or, say, $5\frac{1}{2}$ in. square.

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The thickness of washer $t = \frac{1}{220}\sqrt{\frac{400(30{\cdot}25 - 1{\cdot}5^2) \times 2}{1{\cdot}5}} = 0{\cdot}55$, say, $\frac{1}{2}$ in. thick.

The wale, bolt and washer have been calculated to show the large sizes required. It would be better to use smaller sizes and tie each instead of alternate studs.

The wales can now be made 4 in. by 4 in., as they will only act as stiffeners.

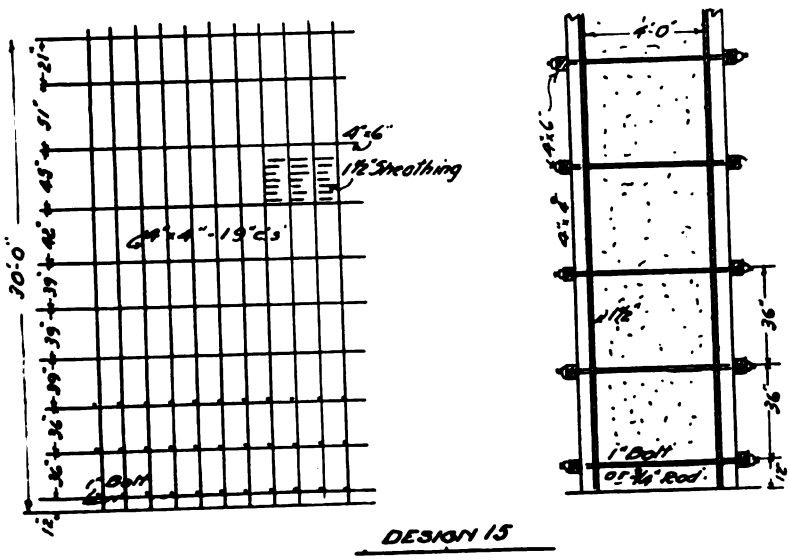
The load now on the tie will be 5600 lbs., and for plain rods

$$d = \sqrt{\frac{5600}{20000 \times 0{\cdot}785}} = 0{\cdot}60 \text{ in., or, say, } \tfrac{5}{8} \text{ in. diameter.}$$

The two lower wales will need $\frac{5}{8}$ in. rods, the third and fourth $\frac{1}{2}$ in. rods, and the fifth $\frac{3}{8}$ in. rods, or two double strands of No. 9 wire and the top one double strand of wire.

DESIGN 15: VERY HIGH WALLS AND PIERS POURED MONOLITHIC.

Design forms for a pier 4 ft. thick and 30 ft. high, no construction joint being allowed.



We will use $1\frac{1}{2}$ in. sheathing, and spacing from Table 2 is 15 in. Using 4 in. by 4 in. studs, they can therefore be placed 19 in. on centre.

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From formula 50, if first wale is placed 12 in. up, spacing of next wale, $s = 66 \sqrt{\frac{3{\cdot}75 \times 3{\cdot}75^3}{75 \times 29 \times 1{\cdot}58}} = 32{\cdot}2$ in.; allowing for the wale and for the fact that we have assumed uniform pressure between wales, say, 36 in. Spacing of third wale, $s = 66 \sqrt{\frac{3{\cdot}75 \times 3{\cdot}75^3}{75 \times 26 \times 1{\cdot}58}} = 33{\cdot}2$, say, 36 in. Similarly the spacing of the remaining wales will be 39 in., 39 in., 39 in., 42 in., 45 in. and 51 in.

		lbs.
Loads brought by studs to lowest wale	$= 29 \times 75 \times 1{\cdot}58 \times 2{\cdot}5$	$=8600$
,, ,, ,, ,, second ,,	$= 26 \times 75 \times 1{\cdot}58 \times 3$	$=9200$

Diameter of threaded bolt $= \sqrt{\frac{9200}{14000 \times {\cdot}785}} = 0{\cdot}91$, say, 1 in. diameter, and for plain rod $= 0{\cdot}76$ in., or, say, $\frac{3}{4}$ in. diameter.

The size of rods or bolts for the other wales are similarly calculated. The wales should be 4 in. by 4 in. at least, or preferably 4 in. by 6 in.

With high walls or piers poured monolithic ties will be required on every stud or very large bolts and washers, which are not economical, will be necessary; wires should not be used.

Sometimes a large bolt is replaced by two smaller bolts, placed one each side of the stud (*see* Design 16).

DESIGN 16: VERY HIGH WALLS OR PIERS POURED IN TWO OPERATIONS.

Design forms for a pier 4 ft. wide and 30 ft. high, the concrete to be poured in two operations, but the forms to be built up the full height.

Using $1\frac{1}{4}$ in. sheathing, for 15 ft. high the spacing of the studs will be $14\frac{1}{2} + 2\frac{1}{2} = 17$ in. Using 3 in. by 4 in. studs, wales will be spaced at 12 in., 36 in., 39 in., 42 in., 45 in., 36 in., 39 in., 42 in. and 45 in., the top wale being 24 in. from the top of the wall.

		lbs. from each stud.
Load on lowest wale	$= 14 \times 100 \times 1{\cdot}42 \times 2{\cdot}5$	$= 4960$
,, 2nd ,,	$= 11 \times 100 \times 1{\cdot}42 \times 3{\cdot}125$	$= 4890$
,, 3rd ,,	$= 7{\cdot}75 \times 125 \times 1{\cdot}42 \times 3{\cdot}375$	$= 4640$
,, 4th ,,	$= 4{\cdot}25 \times 125 \times 1{\cdot}42 \times 3{\cdot}625$	$= 2740$

With ties at alternate studs, load on tie = 9920 lbs., and calculating the size of wale required, as in Design 14, a 4 in. by 6 in. will be required. Instead of using one large tie we will use two small ones, and the load

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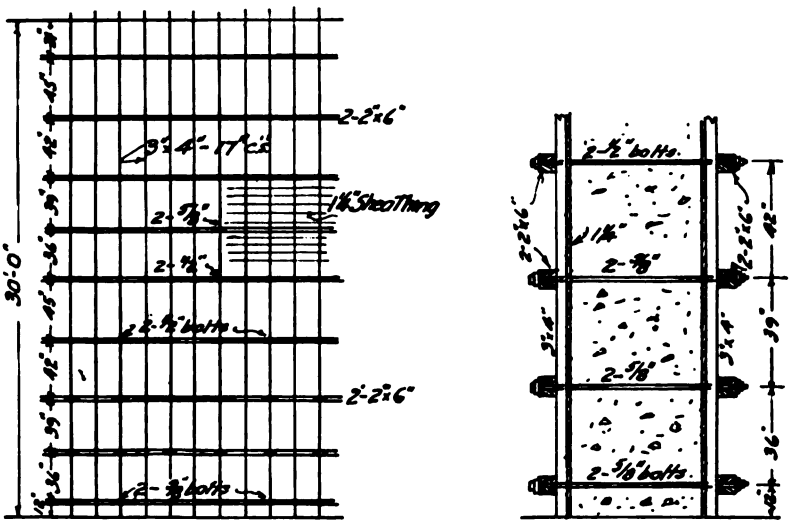
on each will be 4960 lbs.; the diameter of plain rod required is 0·55, or, say, ½ in., or if threaded bolts are used the diameter will be ⅝ in. One rod or bolt will be placed each side of the stud and between the two 2 in. by 6 in., forming the 4 in. by 6 in. wale.



DESIGN 16

Use the same sizes for the three lower wales. For the 4th wale two ½ in. bolts would be sufficient, but it would be better to use the same sizes as before.

The loads on the 5th wale will be greater from the upper pour and (approx.) = 13·75 × 100 × 1·42 × 2·5 × 1·25/3 = 2040 lbs. (exactly 2090). Use ½ in. or ⅝ in. bolts.

The bolts in the wales for the upper half of the wall will be similar to those on the lower half.

DESIGN 17: WALL FORMS WITHOUT INTERNAL TIES.

Design forms for a wall 9 ft. high poured against an existing wall, so that a form for only one side will be required.

When the mass of concrete to be poured is very large, as in bridge abutments, or when a new wall is to be poured against an old one so

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that only one side can be formed, it is impossible to use internal ties, and the forms have to be held by external braces.

With 1 in. sheathing, 3 in. by 4 in. studs will be 16 in. centre to centre, and wales will be placed 12 in., 39 in. and 45 in. up.

lbs.

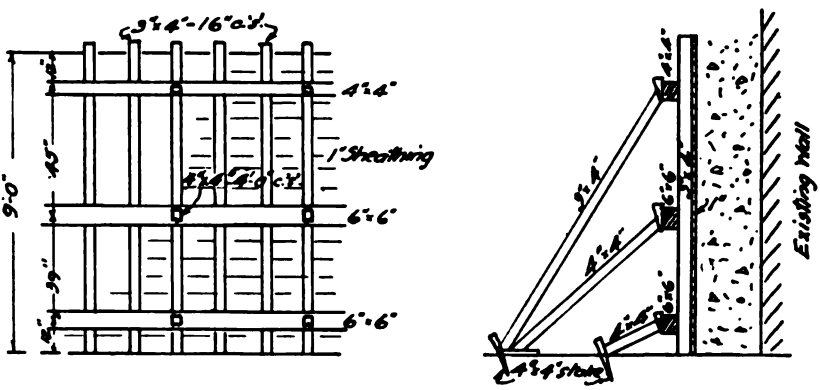
Load on each stud at lowest wale $= 8 \times 125 \times 1{\cdot}33 \times 2{\cdot}625 = 3500$

" " " " 2nd " $= 4{\cdot}75 \times 125 \times 1{\cdot}33 \times 3{\cdot}5 = 2775,$

" " " " 3rd " $= 2{\cdot}875 \times 125 \times 3{\cdot}75 \times \frac{1{\cdot}47}{3{\cdot}75}$

$\times 1{\cdot}33 + \frac{125}{2} \times 1 \times 1{\cdot}33 = 785$

The centre of pressure between the 2nd and 1st wales from the top is 1·47 ft. up from the 2nd wale.



DESIGN 17

Bracing the lowest wale at every third stud or at spaces of 4 ft., to find size of wale required $M = 3500 \times 1{\cdot}33 \times {\cdot}8 \times 12 = M_r = \frac{1600bd^2}{6}$.

or $bd^2 = 168$, or size can be 6 in. by 6 in.

The same size will be used for the middle wale with the braces the same distance apart.

For the upper wale for the same span of 4 ft. a 4 in. by 4 in. wale will be sufficient.

The angle of the brace to the lower wale will be about 30 degrees, and thrust $= 3500 \times 3 \times 1{\cdot}25 = 13125$ lbs. The centre brace will be

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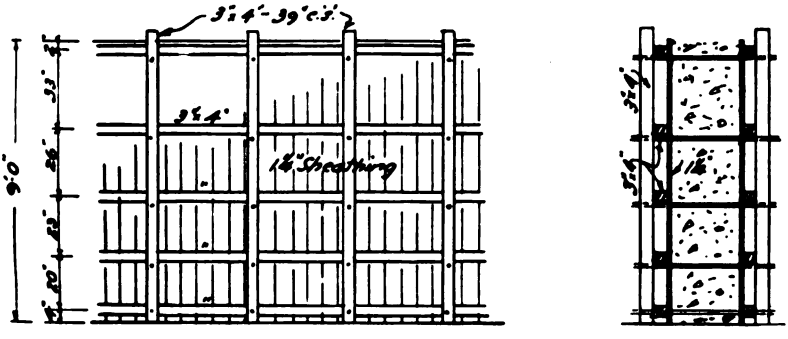
placed at 45 degrees, and thrust $= 2775 \times 3 \times 1{\cdot}5 = 12500$ lbs. The top brace will be at 60 degrees, and thrust $= 785 \times 3 \times 2 = 4710$ lbs. Length of braces will be 2 ft., 4 ft., and 9 ft. respectively.

From Table 11, assuming hardwood wedges are used between brace and wale, 4 in. by 4 in. braces can be used for the bottom and centre wale, and 3 in. by 4 in. braces for the top wale.

DESIGN 18: WALL FORMS WITH HORIZONTAL STUDS.

Design forms for a wall 9 ft. high using $1\frac{1}{4}$ in. sheathing placed vertically.



DESIGN 18

Placing first horizontal stud 4 in. up, from Table 2 or 3 the next spacing will be 17 in. + 3 in. for stud, or 20 in. total; next spacing will be $19\frac{1}{2}$ + say $3\frac{1}{2} = 23$ in.; next spacing will be, from formula 46, for a depth of 9 ft. — (3 ft. 11 in.) = 5 ft. 1 in.,

$$1 = \sqrt{\frac{166{\cdot}67 \times 12 \times 1{\cdot}0625^2}{125 \times 5{\cdot}08}} = 22\tfrac{1}{2} + 3\tfrac{1}{2} = 26 \text{ in.}$$

Similarly, next spacing will be 33 in.

Since we assume that the pressure is constant between any two studs while it decreases rapidly when the spacing is large, it is quite safe to add at least 3 in. to the calculated spacing.

		lbs.
The load on the bottom stud (approx.)	$= 8{\cdot}67 \times 125 \times 1{\cdot}17$	$= 1270$
" " " 2nd " "	$= 7 \times 125 \times 1{\cdot}79$	$= 1570$
" " " 3rd " "	$= 5{\cdot}08 \times 125 \times 2{\cdot}04$	$= 1295$

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The loads on the two upper studs will be less, so the second stud will govern the spacing of the vertical wales or of the tiles.

Using 3 in. by 4 in. studs, to find maximum span for a deflection of $\frac{1}{8}$ in., by formula 14, $\frac{1}{8} = \frac{3 \times 1570 \times 1^4 \times 12^4 \times 12}{384 \times 12 \times 1200{,}000 \times 2{\cdot}75 \times 3{\cdot}75^3}$ or 1 = 3·03 ft., say, 3 ft. without wales and 3 ft. 3 in. with wales.

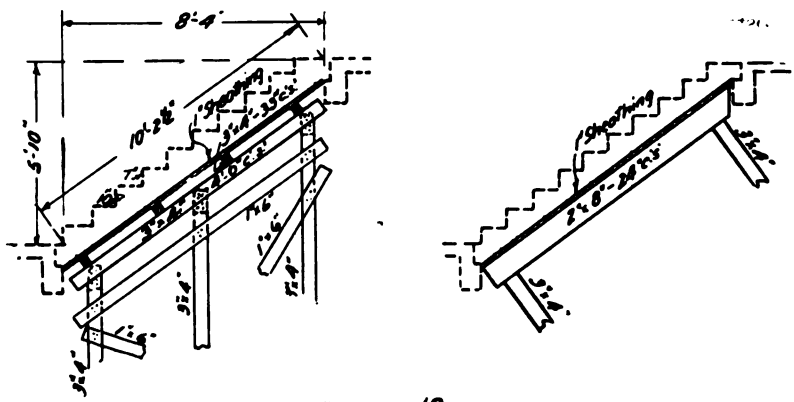
If no wales are used, or if wales are only used to keep the form in line, all the horizontal studs will be tied at this spacing.

If it is desired to take advantage of the smaller loads on the upper studs, short lengths of wale can be used, one from the lowest to the third stud at above spacing and another from the third to the top stud at a greater spacing depending upon the maximum span for the fourth stud. Load on this stud = 2·92 × 125 × 2·46 = 900 lbs. per lin. ft., and calculated as above 1 = 3·5 ft., or, with wales, say, 3 ft. 9 in. As this is not much greater than for the lower wales it would be better to run the wales through to the top at the smaller spacing.

The size of ties is calculated as in previous designs.

DESIGN 19: STAIR FORMS.

Design forms for a flight of stairs 5 ft. 10 in. high from landing to landing 4 ft. wide with 10 in. treads and 7 in. risers, the slab being 5 in. thick.



DESIGN 19

This design is typical for inclined slabs.

Weight of slab 60 lbs., steps 35 lbs., live load, say, 40 lbs., or total of 135 lbs. sq. ft. Not more than 40 lbs. need be taken for live load because there will be no wheeling over the slab.

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The dead load will act vertically, while the sheathing and joists will be inclined so that they will not receive the full load, and the problem is to find the load for which they should be designed ; then the remaining calculations are as before.

When the spans are measured along the incline, the vertical load can be reduced by the ratio of the horizontal run to the inclined span of the stairs (which can be scaled or $= \sqrt{8{\cdot}33^2 + 5{\cdot}83^2}$), or in this case by $\frac{8{\cdot}33}{10{\cdot}2} = 0{\cdot}816$.

Load to be used for designing is then $135 \times 0{\cdot}816 = 110$ lbs. sq. ft.

To make use of Table 1, deduct 75 lbs. for live load leaving 35 lbs. or the equivalent of a 3 in. slab, and span for 1 in. sheathing is $32\frac{1}{2}$ ins. say, 33 in.

The span of the joists will be 4 ft., and using 3 in. by 4 in. from formula 5 $s = \frac{1600 \times 2{\cdot}75 \times 3{\cdot}75^2}{110 \times 16} = 35$ in. Therefore strength of sheathing governs spacing, which will be 33 in.

Load carried to each ledger $= 110 \times 2{\cdot}75 \times 2 = 605$ lbs.

From Table 5, for 605 lbs. at 33 in. centre to centre a 2 in. by 6 in. will span 6 ft., which will be the spacing of the posts. However, with one post at the centre the span will only be [illegible] ft., and 3 in. by 4 in. ledgers can be used.

Posts should be at right angles to the slab, or if they are placed vertical they should be well braced and prevented from kicking over at the top.

If it is desired to run the joists longitudinally without using ledgers the span will be 10 ft. and for $s = 24$ in., $24 = \frac{1600 \times bd^2}{110 \times 100}$ and $bd^2 = 163$, or 2 in. by 8 in. is required.

Posts can be 3 in. by 4 in.

[*To be continued.*]

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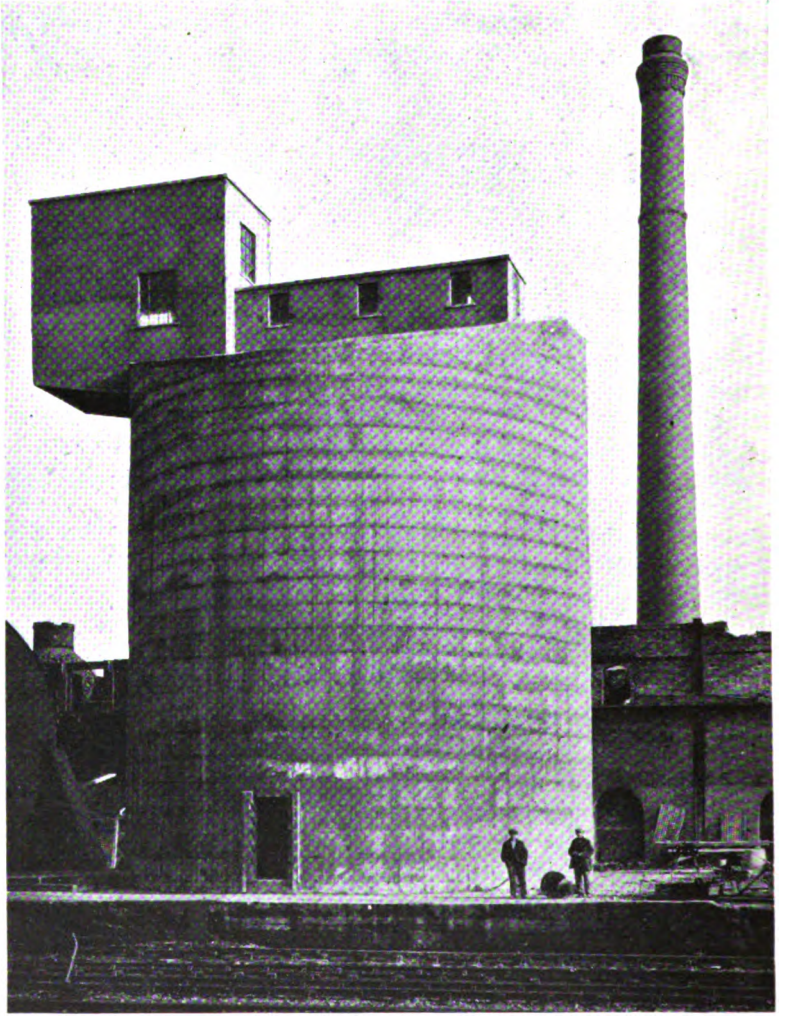

Reinforced Concrete Cement Silo at Arlesey.
(*See p.* 157.)

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REINFORCED CONCRETE CEMENT SILO.

THE silo illustrated herewith has just been completed for the Associated Portland Cement Manufacturers, Ltd., at their works at Arlesey, Bedfordshire, for the storage of cement. Up to the underside of the roof slab the height of the silo is 55 ft., while the external diameter is 42 ft. 6 in., and the capacity 3,000 tons of cement.

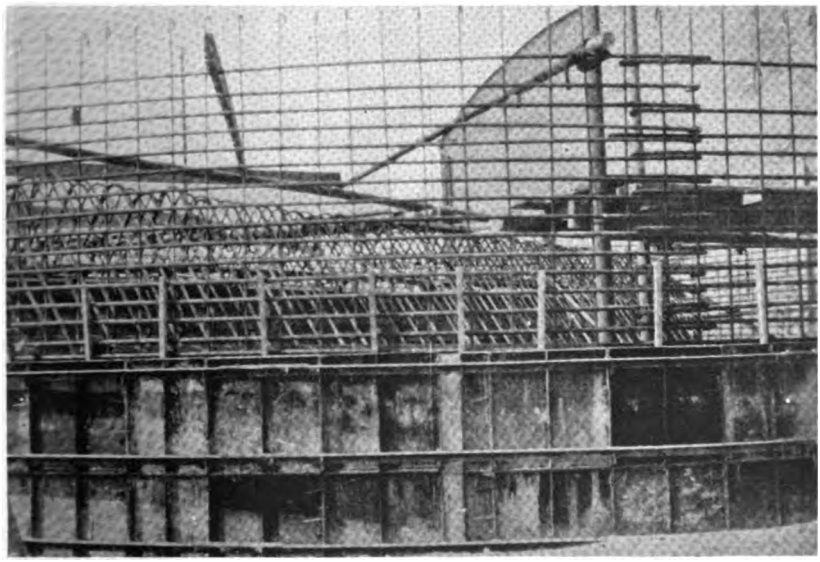
Comprised in the complete scheme there are, in addition to the silo, a machinery pit, a feed-screw chamber on the roof, and an extractor-screw chamber at floor level. Adjoining the machinery pit is an existing cement warehouse, and the wall of the pit, in one part 18 ft. deep, had to be designed strong enough to take the load of this building as well as the weight of the soil retained. During the excavation of the site water was encountered in the marly chalk on which the silo is built. The footings of the walls of the silo, retaining walls, etc., were designed so as not to exceed a load of 2 tons per ft. super on the ground.

The walls of the silo are 12 in. thick at the bottom, tapering to 7 in. at the top, and are reinforced with mild steel bars of various diameters. There are two door openings in the walls at ground level, each 6 ft. high by 3 ft. wide, and at the level of the bottom of the silo is another large opening measuring 7 ft. by 5 ft. The drawing reproduced on pp. 158–9 shows how the tension bars were bent and taken under or over the openings; tension anchor bars were used. The grouping of the several bars of large diameter where they pass over the openings presented some difficulty, and the method by which this was overcome is shown in the detail drawing. The sides of the door openings were grooved to receive 9 in. by 2 in. boards, fixed louvre fashion, and on the outside face of the walls near the doors short lengths of rolled steel joists were fixed to take the 9 in. by 3 in. external timber door-frames.

The bottom of the silo is 8 in. thick, reinforced with a mesh of mild steel bars, the slab being quite independent of the walls and not bonded to them in any way.

The extractor-screw chamber is hopper-shape. The walls of the hopper are 17 in. thick, reinforced, and as there are two lines of extractor screws poke holes and delivery hopper openings occur on both the walls of the chamber.

The roof of the silo, which is partly flat and partly sloping, is carried by two main beams and two secondary beams; the former is cantilevered out over the



REINFORCEMENT TO SILO WALLS, SHOWING STEEL SHUTTERING.

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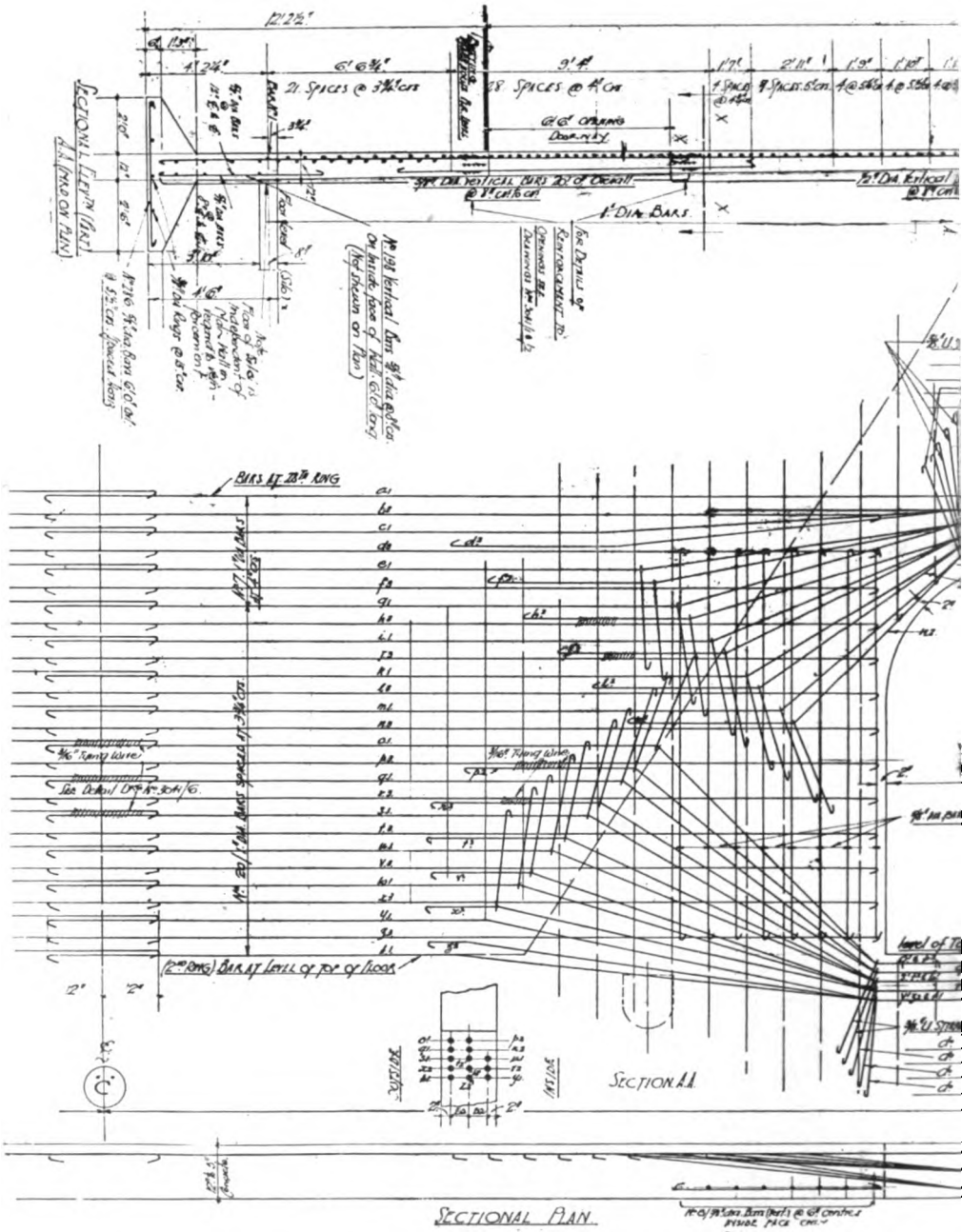

Top : Details of Reinforcement to Main Walls. *Bottom :* Developm

REINFORCED CONCRETE C

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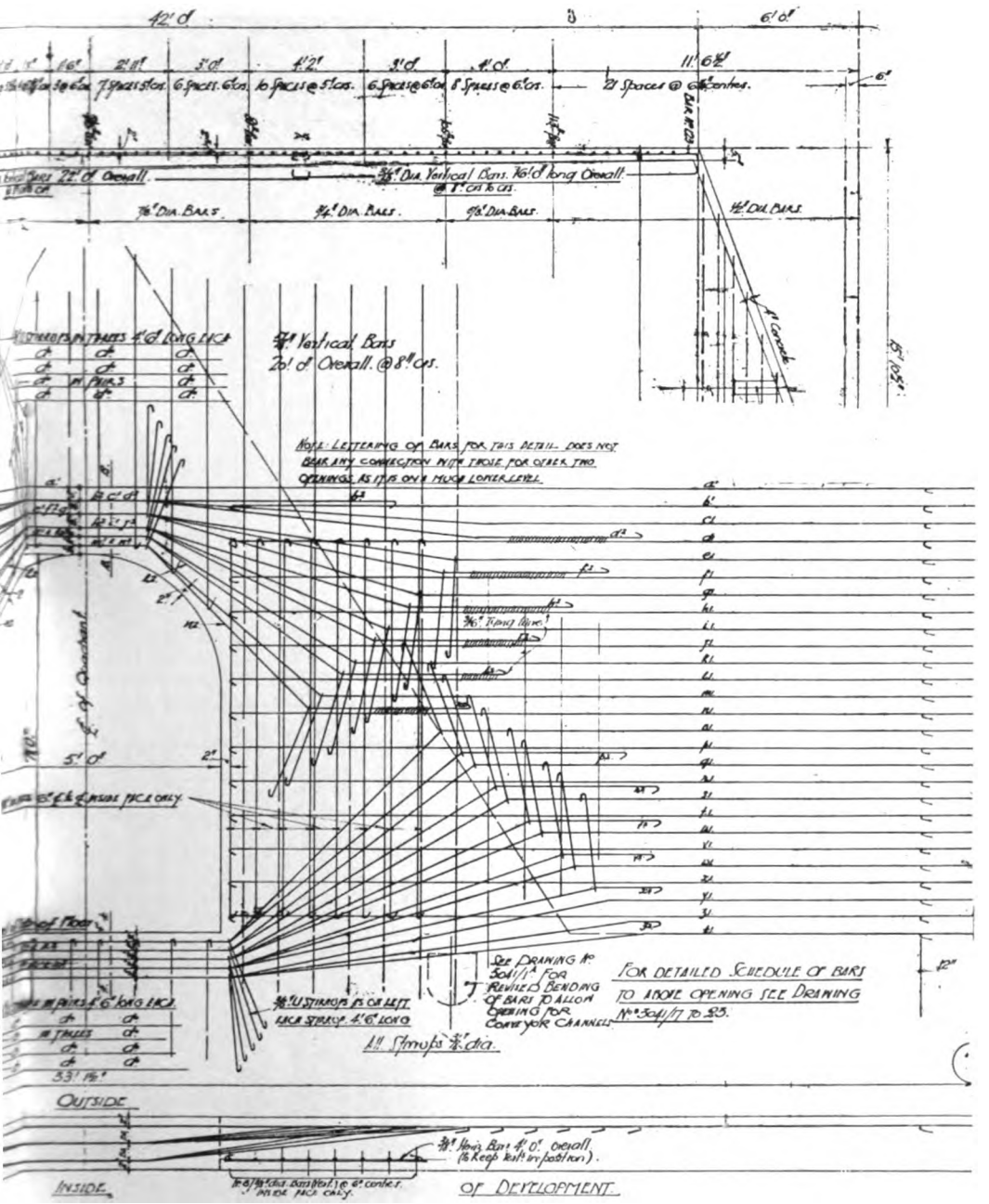

...ENT OF QUADRANT CONTAINING DOORWAY TO EXTRACTOR CHAMBER.

...EMENT SILO AT ARLESEY.

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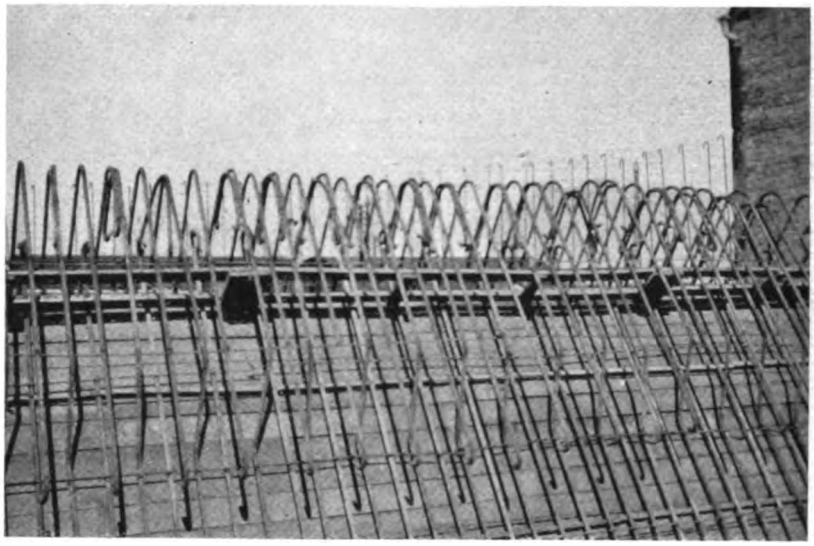
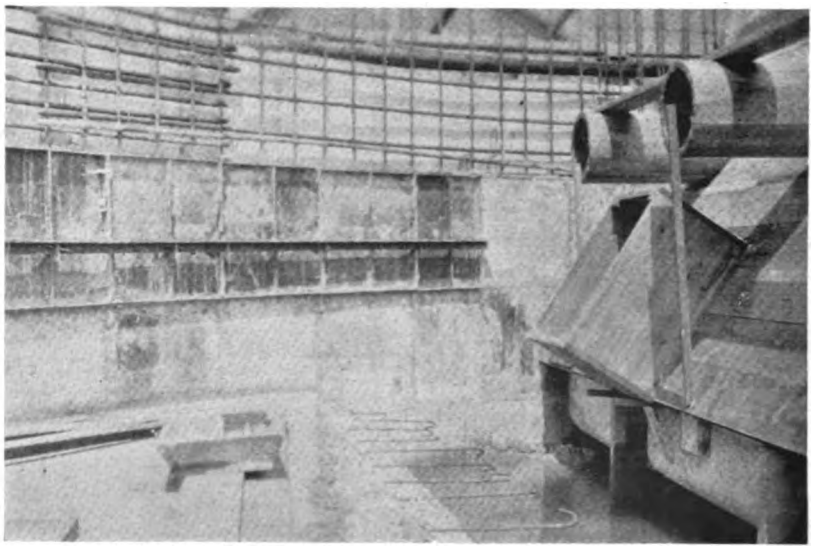


REINFORCEMENT TO EXTRACTOR CHAMBER.



VIEW OF BOXING FOR HOPPERS AND POKE HOLES TO EXTRACTOR CHAMBER.

REINFORCED CONCRETE CEMENT SILO AT ARLESEY.

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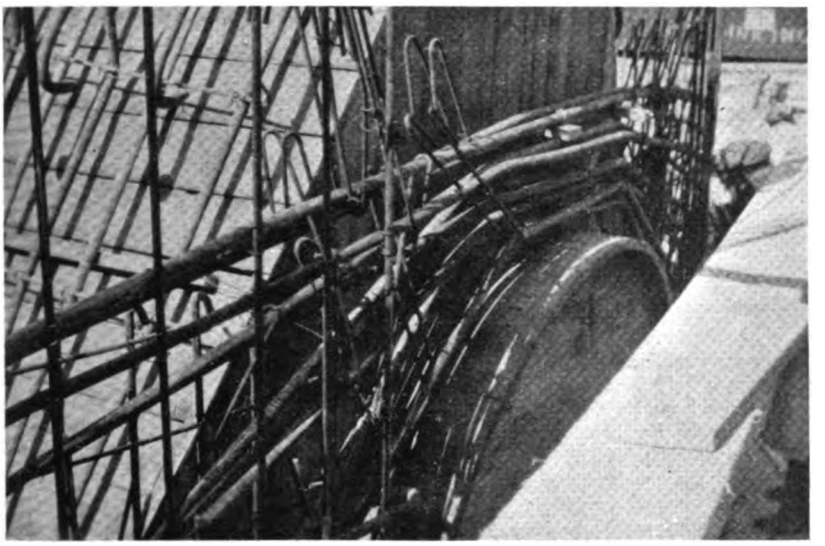

REINFORCEMENT OVER ARCH TO OPENING IN EXTRACTOR CHAMBER.

wall to carry the feed-screw chamber, the projection from the face of the wall being 11 ft. The elevator house and the feed-screw chamber extend over the whole length of the roof. There are three openings in the floor of the feed chamber through which cement is discharged into the silo.

Access to the silo is obtained through a dust-proof trapdoor in the roof, under which wrought-iron ladders are fixed down to the floor below.

In the construction of the walls of the silo use was made of " Blaw-form " steel shuttering, fixed to two courses (4 ft.) high all round the building and concreted to this height in one operation.

Drawings showing the details of the reinforcement to the main walls, and the development of the quadrant containing the doorway to the extractor chamber, are given on pp. 158–9.

The reinforced concrete work was designed by Mr. T. A. Harvey, consulting engineer, on behalf of the London Structural Co., Ltd., who were the contractors for the work.

SHOWING REINFORCEMENT TO SILO WALLS.

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QUESTIONS AND ANSWERS RELATING TO REINFORCED CONCRETE.

Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.—ED.

Reinforced Concrete Piles.

QUESTION,—I shall esteem it a favour if you can give me addresses of firms specialising in the manufacture of shoes for concrete piles, and should be glad of your remarks as to the limits of length for reinforced concrete piles and the best type of aggregate and proportion of mix for such piles.

Is any advantage to be gained by the use of a quick-setting cement, and will concrete made with such stand up under the severe stresses induced in long piles by handling prior to driving ?

Is a pile 75 ft. long by 16 in. or 18 in. diameter (or over flats if octagonal) a structural possibility, and what is the permissible limitation of stress ? It would seem that it might be practicable to make and handle piles of the dimensions stated provided special arrangements were made for lifting and slinging, but stresses up to 1,200 to 1,500 lbs. per sq. in. on concrete would seem to be involved, with about 18,000 to 20,000 lbs. per sq. in. on steel.

It would apparently be necessary to mould the piles on a hollow core to lighten them as much as possible, and a thin steel tube of 9 in. diameter seems to be the limit attainable in an 18-in. pile to ensure a sufficiency of cover to the reinforcement.

Have any piles near the dimensions stated ever been successfully made in reinforced concrete and used anywhere ? If so, I should much appreciate particulars of same. W. G. WATKINS.

ANSWER.—Reinforced concrete piles have been made up to 80 ft. in length, but long piles have to be made correspondingly large in section, otherwise they are liable to break in handling. In the design of a pile the bending moments due to its weight when handling are an important consideration. The best concrete for piles is crushed granite, but in districts where granite is not readily obtained crushed gravel is often used. Good proportions are 7 cu. ft. of ¾-in. crushed stone, 3½ cu. ft. of sand, one bag of cement containing 200 lbs.

The use of a quick-hardening cement is certainly an advantage, as it enables the piles to be handled and driven at an earlier date. Quick hardening is not to be confused with quick setting, and the setting time of the cement should not be less than ½-hour initial set as the concrete cannot then be properly placed in position before the initial set begins.

A pile 75 ft. long would need to be at least 18-in. square, and the stresses during handling should be limited to 600 lbs. per sq. in. in the concrete and 16,000 lbs. per sq. in. in the steel. The high stresses referred to are not advisable, and when proper regard is taken to the compressive resistance of the reinforcement they are not necessary.

Special arrangements should certainly be made for lifting and slinging. It is an advantage to lighten the piles by coring the centre, as suggested. Long reinforced concrete piles are undoubtedly heavy, but have nevertheless been used in many places.

A list of makers of pile-shoes has been sent to our correspondent.

Wire Rope as Reinforcement.

QUESTION.—We should be much obliged if you would answer the following query : Would a ⅞-in. old wire rope cut into the desired lengths make an efficient reinforcing ? P. W. L.

ANSWER.—The wire rope cannot be considered as a suitable substitute for ordinary mild steel rods for reinforcement, although under some circumstances it could be adopted for unimportant work. The chief difficulty will be that of keeping the rope taut and in the correct position during the placing of the concrete, and unless this can be done it will not satisfactorily act in taking up the tension stress immediately this is caused.

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THE FOREMAN'S GUIDE TO CONCRETE.—V.

A MANUAL FOR ALL ENGAGED IN CARRYING OUT CONCRETE AND REINFORCED CONCRETE STRUCTURES.

By ALBERT LAKEMAN, M.S.A., M.I.Struct.E.

REINFORCED CONCRETE BEAMS.

REINFORCED concrete beams are of different types according to the conditions and loads that have to be dealt with by the designer, and it is not always practical to provide the same type of beam throughout a structure. A description of each type is therefore necessary to make it clear why variations take place, and the importance of the materials used.

Single Reinforced Beams.

The simplest and most economical type of beam is that known as the "single reinforced," where the main steel rods are all placed in the bottom portion of the beam to give resistance to tension, and the beam is simply supported at the ends and not fixed in any way. The difference between supported and fixed beams will be dealt with later on, and for the present the notes will cover those supported only.

An example of a single reinforced concrete beam of rectangular shape is given in *Fig.* 29. It will be seen that the total depth is greater than the "effective" depth, and this is an important point. The depth of the beam is one of the factors that affect the resistance, and when making the calculations the designer works to a fraction of an inch in order to produce an economical beam. As the concrete below the neutral axis is neglected, however, as previously explained, the depth used by the designer is the distance from the top of the beam to the centre of the reinforcement; the concrete below this is provided as a protection to the steel only, and does not add to the resisting power of the beam when supporting a load.

The strength of a beam will vary according to the width, the depth, and the reinforcement provided, and a variation in any of these during the execution of the work will mean that the strength provided for by the designer will not be given if the variation has resulted in any one factor being less than calculated for. A variation in the width can be found by measurement, and this, together with the total depth, can be checked by testing the dimensions of the form work before concreting or actual measurement of the beam when the forms are removed, but the amount and position of the steel cannot be checked in this manner. The omission of one of the steel rods is, of course, possible, but generally speaking the ordinary care exercised will prevent this occurring, as the foreman should realise the serious consequences likely to arise from lack of supervision in this respect.

The position of the reinforcement, however, is not always considered as such an important matter, and in many cases a variation of an inch or more will be overlooked. It will be clearly seen that if the steel is placed farther from the bottom

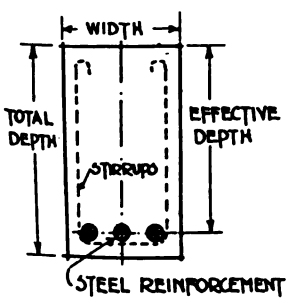

FIG. 29.

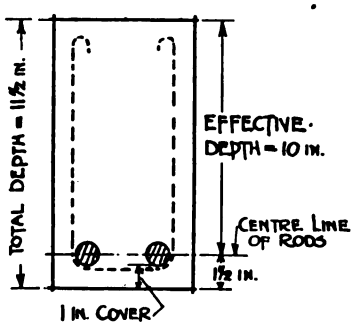

FIG. 30.

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of the beam than shown on the drawings the effective depth is reduced, and consequently the resistance of the beam is decreased; although the total depth of the concrete may be correct. As a simple example, suppose the designer in his calculations finds that the load requires a beam having an effective depth of 10 in. and the reinforcement required is two 1 in. diameter rods. Then the section of the beam required will be as shown in *Fig.* 30, where the effective depth is given from the top surface to the centre of the rods, and, allowing for the minimum cover of concrete of 1 in. to the rods, the total depth will be 11½ ins. Assuming that the form work is correctly made and the two rods are placed in the form but owing to carelessness the reinforcement is not put, or kept, at the right level, and instead of being 1 in. from the bottom of the beam it is 1 in. too high, giving a distance of 2 in. from the bottom, then the effective depth is reduced from 10 in. to 9 in. This may not seem a serious variation at first sight, but it will become clear when it is realised that one of the factors used by the designer when making the calculations has been reduced by one-tenth, and the beam weakened accordingly. The material provided for in the design will all be used, the beam will be of the correct size when measured, yet the strength necessary according to the calculations will not be present because the effective depth has been reduced.

In addition to the main steel reinforcement shown in *Fig.* 29, it will be noticed that stirrups are indicated, and these are provided to resist the horizontal shear and to anchor the main rods to the concrete on the top portion of the beam. The effect of horizontal shear has been previously dealt with, and it will be observed that these stirrups must be cut through by the shearing action before the concrete can fail by sliding in horizontal layers.

When a single reinforced concrete beam is designed in an economical manner to carry an ordinary distributed load, and the designer is not limited to depth and width, the concrete provided will always be sufficient, theoretically, to give the necessary resistance to shear and no vertical stirrups will be required according to the calculations. Some anchorage for the main steel rods must, however, be provided to prevent any tendency for these rods to spring downwards and break away from the concrete above, and for this reason stirrups must always be included in the beam. This is dealt with in the Regulations of the London County Council, where it is stated that the shear or web reinforcement, as the stirrups are called, shall be spaced according to the requirements of the shearing stress, but in any case the stirrups must not be spaced apart at a distance which exceeds a length equal to the arm of the resistance moment. The arm of the resistance moment means the distance from the centre of the steel rods in the lower part of the beam to the centre of the compression resistance as shown in *Fig.* 31. The shaded triangle in this diagram indicates the concrete above the neutral axis, which is theoretically capable of offering resistance of 600 lbs. per sq. in. to the compression stress, and the centre of the resistance will therefore be situated at a

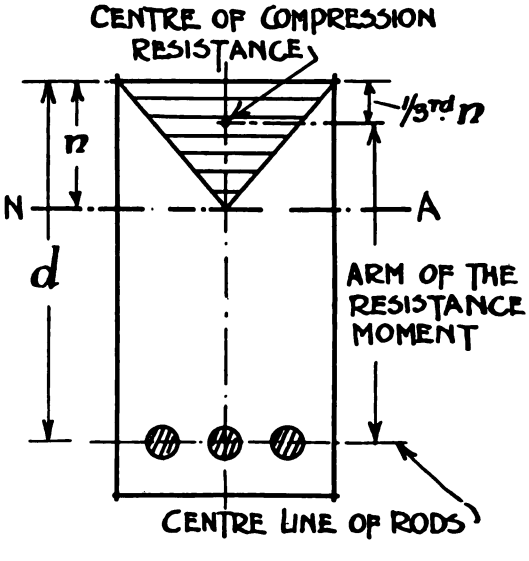

FIG. 31.

distance of one-third of the depth from the top surface to the neutral axis. In other words, if n equals the depth from the top surface to the neutral axis, and d equals the effective depth, then the arm of the resistance moment, or the lever arm, equals d minus one-third of

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n. This is written in calculations as $d-\frac{n}{3}$. In no case must the stirrups be omitted or spaced at a greater distance apart than shown on the drawings, because the anchorage of the steel rods will not then be properly provided for even if the stirrups are not wanted for shear resistance.

Double Reinforced Beams.

It will be necessary to give some additional notes on single reinforced beams as regards the variation of the stresses and other matters, but as these notes will also apply to other types of beams they will be referred to after describing the different types.

In many cases the designer is unable to provide a satisfactory single reinforced concrete beam owing to heavy loads which require considerable resistance, and the amount of concrete necessary would result in a beam which becomes excessive as regards size, especially if the available headroom is restricted. In order to reduce the amount of concrete and allow the beam to be kept within practical limits the resistance to compression is provided by the concrete above the neutral axis assisted by some reinforcement in the compression area. The beam is then reinforced in both the bottom and top surfaces, and is known as a double reinforced beam. A section of this type of beam is shown in *Fig.* 32. It will be seen that the percentage of steel employed is greater than that necessary with a single reinforced beam, and as the steel is the expensive material the double reinforced type is not so economical as regards cost of execution.

It was previously mentioned that where steel reinforcement is used to resist compression the permissible stress is limited to fifteen times the stress on the surrounding concrete, and in this case it means that the stress is limited to something less than 600 × 15 lbs. per sq. in., which is obviously not an economical use for the steel, as the full strength cannot be developed. The maximum stress of 600 lbs. per sq. in. on the concrete can only be considered at the extreme top surface and this gradually diminishes toward the neutral axis as previously explained, so the steel cannot be placed in the layer when the maximum compression occurs, unless it is situated right on the top surface. The rods must, however, be covered with concrete as a protection, and must therefore be kept down sufficiently to

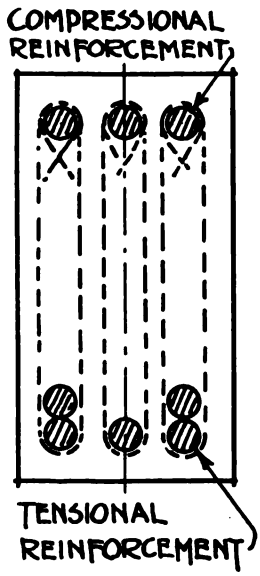

FIG. 32.

give at least 1 in. between the top of the rod and the top of beam. The maximum stress being 600 lbs. per sq. in. on the concrete, and this being reduced to nil at the neutral axis, the layer where the steel is placed cannot be stressed to the full amount.

As a simple example, assume that the depth from the top of the beam to the neutral axis is 6 in., and the stress at the top surface is 600 lbs. per sq. in. If this is the stress at a distance of 6 in. from the axis, then the stress at a distance of 5 in. will be 500 lbs., at 4 in. it will be 400 lbs., and so on. A cover of 1 in. of concrete over steel rods, say, 1 in. in diameter, will mean that the centre of the reinforcement will be 1½ in. from the top of the beam, or 4½ in. from the neutral axis, and the stress on the concrete at this point must not be more than 450 lbs. per sq. in. The maximum stress on the steel in this position must not exceed fifteen times this amount, which equals 450 × 15 = 6,750 lbs. per sq. in., and it will be realised that as steel will safely resist 16,000 lbs. per sq. in. the best value as regards economical use is not being

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obtained from the material. This explanation covers the above remark that the stress will be something less than 600 × 15 lbs per. sq. in. in the case of steel used in the compression area of a double reinforced concrete beam.

Where very heavy loads have to be carried the low limit of stress on the compression sometimes results in more steel being wanted in the top portion of the beam than in the bottom portion, although the concrete above the neutral axis will help in resisting the load. This limitation of the stress that can be put on the steel is frequently so serious that the allowance for the resistance of the concrete becomes a disadvantage because the limit on the steel is fixed by the safe stress on the concrete. The only way to overcome this disadvantage is to neglect entirely the resistance of the concrete to compression and provide sufficient steel to take the whole of the compression stress in a similar manner to the method adopted for the tensional resistance; because if this is done then the safe stress on the steel in compression can be taken as 16,000 lbs. per sq. in. A large number of double reinforced beams are designed in this manner, and generally speaking this method will be found more economical than that where the concrete is included in the resistance, if the loads are very heavy.

When the calculations are based on a stress of 16,000 lbs. per sq. in. on the steel in compression, it is necessary to neglect the strength of the concrete, because theoretically the concrete will be crushed before the steel can develop this amount of resistance. In actual structures the crushing of the concrete is not, of course, visible to the naked eye, but the crushing would result in numerous tiny cracks throughout the material which are not a serious disadvantage from a practical point of view.

It is necessary to anchor the compression steel in position, and sufficient stirrups must be provided for this purpose. The regulations of the L.C.C. state that where the steel in compression is stressed to 16,000 lbs. per sq. in. it must be anchored by anchors passed round or hooked over both the compressive and tensile reinforcement, and these anchors must not be spaced more than 6 in. apart centre to centre. The anchors are required both vertically and laterally, and this means a considerable number of links throughout the length, which adds to the cost of the work and also makes the execution more complicated.

Tee-Beams.

The third type of beam used in reinforced concrete design is that known as the tee-beam, and this is so-called because the section is "T" shape.

It has been explained that it is difficult to provide sufficient resistance to compression in the ordinary type of beam where heavy loads have to be carried, and in order to overcome this difficulty the beam is designed as a tee-section with a wide flange to give sufficient concrete in the top part of the beam, as shown in *Fig.* 33. It will be seen that the top or compression flange is much wider than the main beam itself, and this flange is provided by the floor slab which forms the panel between this beam and the adjoining ones, the beam acting as a rib and containing the tensile reinforcement. This is shown more clearly in *Fig.* 34, which indicates a section through a portion of a reinforced concrete floor where the slabs and beams are given; the portion which is hatched shows how the tee-beam section is provided by the beam and slab, which act together and can be calculated as a tee-shape member. The width of the slab that can be calculated as the flange of the tee-beam is limited

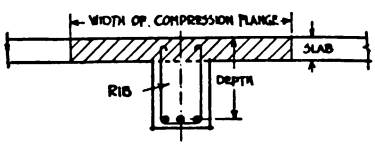

FIG. 33.

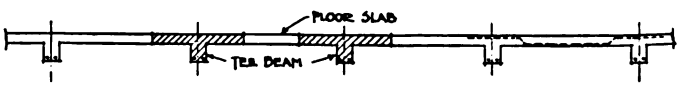

FIG. 34.

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by the regulations of the L.C.C., and it must not exceed either of the following:

(1) One-quarter of the effective span of the tee-beam;

(2) The distance between the centres of the beams; or

(3) Twelve times the thickness of the slab.

As an example, assume that a floor is constructed with beams having an effective span of 20 ft. and spaced at 6 ft. centres, while the thickness of the floor slab is 6 in., then the width of the flange must not exceed (1) one quarter of 20 ft. which equals 5 ft., or 60 in., (2) the distance between the centres of the beams, which is 6 ft. or 72 in., or (3) twelve times the thickness of the slab which equals 6 in. × 12 in. = 72 in. As the least of these three factors is the first mentioned this will fix the limit, and in this case the flange considered in the calculations must not be wider than 60 in.

The position of the neutral axis in a tee-beam can be found by calculation, but this is too complicated to be dealt with here, and the only general note that need be given is a reference to the variation of the position. In some beams the neutral axis will be found to come within the thickness of the slab which forms the top flange, in others it will occur at the junction of the slab with the rib of the beam, while in many cases it will be situated below the slab and in the rib itself. When the axis occurs in the rib itself the concrete between the underside of the slab and the axis is so small, and the value so limited, on account of its nearness to the axis, that its resistance value is almost negligible, and for all practical purposes it can be considered that the whole of the compression is taken by the flange. The reinforcement in the slab which is at right angles to the length of the beam must be carried right across the full width of the portion that is acting as a flange to the beam.

Ell-Beams.

In some cases beams are required which have a slab on one side only, as for example those acting as the trimmers around an open well for a staircase, lift, or similar purposes, and it is not possible to design the beams with a tee section. If it is necessary to include the slab in the compressive resistance calculations the beam becomes an ell-section, as shown in *Fig.* 35, and the compression flange is indicated by the hatched portion. The width of slab that can be included as part

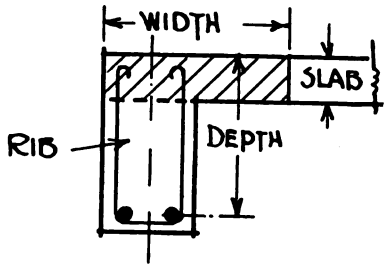

FIG. 35.

of the beam is limited to four times the thickness of the slab for all ordinary cases and must not exceed six times the thickness when special reinforcement is provided to deal with the complicated stress that is set up by the one-sided flange section.

It will be seen from these notes that the design of an ordinary reinforced concrete floor is not quite as simple as might appear at first sight, because apart from any question of supported or fixed ends, or concentrated loads and special trimming, the designer must calculate and produce the drawings for a combination of slabs and beams which will be economical as regards materials and cost while considering questions of head-room and appearance. The spans must be kept within a reasonable limit for slabs and beams, and the question of the most suitable type of beam must be settled both for secondary and main beams where these are employed. The floor slabs will, in all ordinary cases, be single reinforced, and no stirrups or other provision for shear will be necessary, while the beams may be any one of the types given above.

It will be realized that where tee-beams are provided it is essential for the concrete composing the rib and the flange to be well executed with no weak line between the two parts, because they are calculated as one member, and the slab and rib must act together. It is also very important that the thickness of the slab as given in the drawings be accurately formed, as any reduction in this will affect not only the strength of the slab but also the resistance of the beam.

Beams supporting walls can generally

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be designed as the single reinforced type because the necessary depth can be obtained by extending the beam above the floor into the wall, and where the walls are also of reinforced concrete a portion of the wall is designed as a beam and thus no difficulty is experienced in providing the resistance required.

The necessity for uniformity of good work can be realised from these notes on the types of beams, as it will be seen that the various parts of a reinforced concrete structure are not designed as separate members but advantage is taken of the nature of the material to make one part of the construction assist other adjoining parts, and unless the work is well executed throughout a weakness in one section will seriously affect other portions.

(*To be continued*)

Shuttering Competition.

THE Minister of Health has accepted the recommendation of the Committee on New Methods of House Construction, presided over by Sir Ernest Moir, Bt., M.Inst.C.E., that a prize should be offered for the best and most economical system of shuttering or equivalent suitable for use in connection with poured or *in situ* concrete cottages, and has arranged for the sum of £500 to be available for this purpose. A prize of £250 will be awarded for the system of shuttering adjudged by the Committee on New Methods of House Construction to be the best submitted, and the remaining sum of £250 may be awarded in additional prizes at the discretion of the Committee.

In judging the shuttering, special consideration will be given to the following points :—

Economy; the production of a satisfactory finish on the external and internal faces of the wall with the minimum of further treatment ; the ease with which the shuttering can be fixed, moved and refixed by other than skilled men ; durability in use ; facility for use with different designs of cottages ; adaptability for use with cavity walls, etc.

In the first instance competitors will be required to fill in a form stating that they are entitled to exhibit the system, and whether it is the subject of any patent. With this form they will submit drawings and a short description setting out the nature of the shuttering, the materials of which it is composed, and the method of its use. From the proposals thus submitted the Committee will select the most promising, and the proposers of the selected methods will be required to submit samples of the shuttering or large scale models. Before the final selection is made the Committee may, and probably will, require a number of the methods to be subject to test in actual use under working conditions, and the competitor will be expected to provide the necessary shuttering for such test at his own expense.

Only new methods will be considered by the Committee, and methods which are already in use or for which patent rights had been applied for before January 1, 1925, will not be considered.

Applications should be sent to the Secretary, Ministry of Health, Whitehall, S.W.1, from whom forms can be obtained, and must reach the offices of the Ministry not later than May 31, 1925.

Trade Notices.

A REPORT, compiled from information furnished by His Majesty's Chargé d'Affaires at Santo Domingo (Mr. Darrell Wilson), regarding the increasing demand in the Dominican Republic for twisted reinforcing concrete rods and galvanised sheets, has been issued by the Department of Overseas Trade, to firms entered on its special register. Other United Kingdom firms desirous of receiving a copy of this report should apply to the Department of Overseas Trade, 35 Old Queen Street, London, S.W.1, quoting reference A.X. 1697.

WE are informed by Messrs. Builders & Contractors Plant Ltd., that the concrete mixer shown in operation in one of the illustrations in our last issue in connection with the new race tracks in France is one of their " Roll " centrifugal mixers.

THE concrete roads at Cardiff, referred to in our February issue, are reinforced with B.R.C. fabric, supplied by the British Reinforced Concrete Engineering Co., Ltd.

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	"	14	0
" " ½ in. mesh	"	15	0
Best British Portland Cement	per ton 58s. to	63	0
"Ferrocrete" Portland Cement	delivered London 10s. per ton extra		
"Super-Cement"	d/d 4 miles Charing Cross, per ton 88s. to	93	0

BOARDING FOR SHUTTERING—		Sawn. s.	Sawn. d.	Wrot. s.	Wrot. d.
1 in.	per square	23	6	27	6
1¼ in.	"	29	6	33	6
1½ in.	"	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in.	from £24 per standard
3 in. by 6 in. and 3 in. by 7 in.	" £23 " "

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	0
7/16 in. to ½ in. Rounds	"	13	6
⅜ in. Rounds	"	14	6
¼ in. Rounds	"	15	6

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED WITH CLEAN WATER IN PROPORTION 1 : 2 : 4—

			s.	d.
Do.	do.	in foundation	per yard cube	48	6
Do.	do.	in columns	" "	53	6
Do.	do.	in beams	" "	53	6
Do.	do.	in floor slabs 4 in. thick	per yard super	5	8
Do	do.	in floor slabs 6 in. thick	" "	8	5
Do.	do.	in floor slabs 9 in. thick	" "	12	3
Do.	do.	in walls 6 in. thick	" "	8	8

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—

		s.	d.
From ¼ in. to ⅝ in.	per cwt.	25	0
" 7/16 in. to ½ in.	"	24	0
" ⅝ in. to 2⅞ in.	"	23	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1½*d.*; ⅝-in. rods, 1¾*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2½*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.*

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.*

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	" "	1	1½
Do. do. to sides and soffits of beams average 9 in. by 12 in.	" "	1	1½
Do. do. as last, in narrow widths	" "	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	" "	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Carpenters working on old shuttering, 1/10½; Labourers on building works, 1/4½; Operators on concrete mixers and hoists, 1/5½; Barbenders, 1/5½.

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Prospective New Concrete Work.

ASHTON-UNDER-LYNE.—*Houses.*—The T.C. has approached the M.H. for permission to erect 200 houses.

BEDLINGTON.—*Houses.*—The U.D.C. has applied for a grant of £35,100 for the construction of 78 houses.

BILLINGHAM. — *Houses.* — The U.D.C. has applied to the M.H. for sanction to borrow £17,170 for the construction of 30 houses.

BLAYDON.—*Concrete Houses.* — The U.D.C. has approached the M.H. for permission to erect six concrete houses.

BLAYDON.—*Houses.*—The U.D.C. has applied to the M.H. for permission to erect 200 houses at Bleach Green.

BLOFIELD.—*Houses.*—The R.D.C. is considering the question of erecting 70 houses.

BUSH.—*Reservoir.*—The Gwyrfai U.D.C. has applied for sanction to borrow £1,800 for the construction of a reservoir at Bush.

CLYDEBANK. — *Concrete Houses.* — The T.C. proposes to erect 1,500 concrete houses, at an estimated cost of £550 each.

COVENTRY.—*Houses.*—The Corporation has a scheme on hand for the construction of 503 houses.

DARFIELD. — *Concrete Bridge.* — The U.D.C. proposes to replace the existing bridge over the River Dove with a reinforced concrete structure.

DUNDEE—*Concrete Houses.*—The Corporation proposes to erect 200 concrete houses.

ELTHAM (LONDON).—The Woolwich B.C. is to erect experimental concrete houses at Eltham.

EMLY (IRELAND).—*Railway Station.*—A scheme has been put forward for the reconstruction of the Emly Railway Station in concrete.

FAILSWORTH. — *Bridge.* — The R.D.C. has approached the County Council with a view to the construction of a bridge over the Rochdale Canal, at an estimated cost of £15,414.

FARSLEY (YORKS.). — *Houses.* — The U.D.C. is considering a scheme for the construction of 30 houses.

FRIMLEY.—*Houses.*—The U.D.C. has approached the M.H. for a loan of £48,000 for housing purposes.

GOOLE.—*Houses.*—The U.D.C. proposes to erect 200 houses.

GRAVESEND. — *Houses.* — The Corporation is discussing a scheme for the erection of 200 houses.

GREENOCK. — *Concrete Houses.* — The Corporation is inquiring into the cost of erecting 200 concrete houses.

GRIMSBY.—*Footpaths.*—The T.C. has applied for a loan of £1,450 for flagging footpaths.

HANLEY. — *Houses.* — The Stoke-on-Trent Corporation proposes to erect 750 houses at Hanley and on the Meir site.

HEBBURN.—*Houses.*—The U.D.C. proposes to build 100 houses at Black Road.

HEMINGTON. — *Houses.* — The Frome R.D.C. proposes to build 20 houses at Hemington.

HONLEY.—*Road.*—The U.D.C is considering the question of laying a road at Upper and Lower Westcroft.

HURST.—*Houses.*—The Housing Committee has recommended the D.C. to apply for sanction to erect 200 houses.

KEIGHLEY. *Bridge.* — A scheme is being considered for the erection of a bridge over the River Aire at Keighley, at an estimated cost of about £40,000.

KETTERING. — *Houses.* — The U.D.C. has applied to the M.H. for sanction to borrow £24,500 for the construction of 50 houses.

LETHERINGHAM. — *Bridge.* — The Plomesgate R.D.C. is considering the construction of a bridge at Sanctuary Watering, estimated to cost £1,200.

LONDON (BECONTREE). — *Concrete Houses.*—The L.C.C. proposes to erect 2,000 concrete houses at Becontree, at an estimated cost of £1,200,000.

LOWESTOFT.—*Groyne.*—The Corporation has applied for a grant towards the construction of a groyne adjoining the South Pier.

LYME REGIS.—*Groyne.*—A special committee appointed by the T.C. is discussing a proposal for the erection of a groyne east of the drill hall.

MAIDENHEAD. — *Concrete Houses.* — The T.C. is considering the question of erecting 100 concrete houses.

PLYMOUTH.—*Houses*—The Corporation has housing schemes under consideration to the extent of £1,500,000.

POOLE.—*Sea Defence Works.*—The T.C. is considering proposals for the construction of a sea wall at Canford Cliffs, estimated to cost £20,000.

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PRESCOT.—*Houses.*—The Corporation Housing Committee proposes to erect 100 houses by direct labour.

PRESTON.—*Concrete Houses.*—The T.C. proposes to construct 96 concrete houses on the Ribbleton site, and has asked the M.H. for permission to build 350 houses.

RHONDDA.—*Houses.*—The U.D.C. has applied for a grant of £63,000 for the construction of 110 houses.

SCARBOROUGH. — *Bridge.* — The Corporation is considering a scheme for the construction of a bridge over the ravine near Peasholm Park.

SHREWSBURY.—*Road.*—The Corporation proposes to construct a road from Pengwerne Road to Copthorne.

SKIPTON.—*Houses.*—The U.D.C. is considering a scheme for the erection of 118 houses.

WALLSEND.—*Houses.*—The Corporation has applied to the M.H. for permission to erect 72 houses.

WHITEHAVEN.—*Road.*—The T.C. has applied to the Unemployment Grants Committee for assistance towards the construction of a road from Ginns to the High Road.

WORCESTER.—*Water Tank.*—The City Council proposes to construct a 100,000-gallon water tank, at an estimated cost of £3,500.

YEADON.—*Houses.*—The U.D.C. proposes to erect 70 houses on the Hawthorn site.

YEOVIL.—*Houses.*—The T.C. is negotiating for the purchase of land for the erection of 200 houses.

YORK.—*Road.*—The City Council proposes to lay a road to connect Knavesmire Road and Bishopthorne Road, at an estimated cost of £9,200.

Tenders Accepted.

ARNOLD (*Notts.*). — *Sewer.* — The U.D.C. has accepted the tender of Mr. H. Sewell, Ickeston, at £620 13*s.* 6*d.*, for the construction of about 370 yds. of 24-in. concrete pipe sewer.

BARKING.—*Paving.*—The U.D.C. has accepted the tender of the Patent Victoria Stone Co., Ltd., at 6*s.* 2*d.* per sq. yd., for the supply of 800 sq. yds. of artificial stone paving slabs.

BLYTH AND CUCKNEY.—*Sewage Disposal Works.*—The R.D.C. has accepted the tender of Messrs. Lane Bros., Mansfield, at £20,999 18*s.* 6*d.*, for the construction of sewage disposal works.

BOSTON.—*Concrete Bridge.*—The R.D.C. has accepted the tender of the Yorkshire Hennebique Contracting Co., Ltd., at £3,470, for the construction of a reinforced concrete bridge at Chapel Hill.

BURSLEM.—*Concrete Tank.*—The Stoke T.C. has accepted the tender of Mr. T. Godwin, Hanley, at £1,940, for the construction of a reinforced concrete tank.

CAMBRIDGE.—*Concrete Houses.* — The T.C. has accepted the tender of Messrs. G. Longden & Sons, at £15,810, for the construction of concrete houses at Selwyn Road.

GUILDFORD.—*Concrete Wall, etc.*—The T.C. has accepted the tender of Messrs. W. Stevens & Sons, at £1,240, for the construction of a concrete wall.

HASLEMERE.—*Concrete Houses.*—The U.D.C. has accepted the tender of the Triangular Construction Co., Ltd., East Molesey, at £9,344, for the construction of 20 concrete houses.

HOUNSLOW. — *Paving.* — The Heston and Isleworth U.D.C. has accepted the tender of the Cliffe Hill Granite Co., Ltd., at 5*s.* 7*d.* per sq. yd., for the supply of 6,600 sq. yds. of 2-in. artificial stone paving.

HOVE.—*Concrete Pier.*—The Corporation has accepted the tender of Messrs. Armstrong, Whitworth & Co., for the construction of a 1,250-ft. concrete pier.

HULL.—*Sub-Station.*—The Corporation has accepted the tender of Messrs. Quibell & Son, Ltd., Hull, at £5,336, for reinforced concrete work in connection with the construction of an electricity station.

KETTERING. — *Concrete Pipes.* — The U.D.C. has accepted the tender of the Stanton Ironworks Co., Ltd., at £588 1*s.* 6*d.*, for the supply of 21-in. 18-in. and 15-in. reinforced concrete pipes.

KING'S LYNN. — *Concrete Bridge.*— The Norfolk C.C. has accepted the tender of Messrs. Peter Lind & Co., London, at £9,220, for the construction of a reinforced concrete bridge over the River Ouse at Stow.

LEWISHAM (LONDON).—*Concrete Pipe.* —The B.C. has accepted the tender of

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Messrs. Sharp, Jones & Co. for the supply of about 270 ft. of 15-in. concrete pipe, at 2*s.* 10*d.* per foot.

LONDON (STREATHAM).—*Path and Wall.*—The L.C.C. has accepted the tender of Messrs. R. H. Neal & Sons, at £260, for laying a concrete path and constructing a concrete wall round the Lower Pond, Streatham Common.

LONG EATON.—*Concrete Bridge.*—The U.D.C. has accepted the tender of Mr. E. E. Stevens, Long Eaton, at £5,314, for the construction of a reinforced concrete bridge.

MALTON.—*Concrete Bridge.*—The East Riding C.C. has accepted the tender of the Yorkshire Hennebique Contracting Co., Ltd., Leeds, at £1,215, for the construction of a footbridge at Malton.

MANCHESTER.—*Concrete Bridge.*—The Corporation has accepted the tender of Messrs. Callender's Cable & Construction Co., Ltd., for the construction of a reinforced concrete bridge at Stretford.

MANCHESTER.—*Sheds.*—A contract for the construction of reinforced concrete sheds for the Manchester Ship Canal Co. has been placed with Messrs. B. Morton & Co., Ltd.

MARGATE.—*Concrete Sewer.*—The T.C. has accepted the tender of Messrs. T. Muirhead & Co., Ltd., at £9,857 5*s.* 6*d.*, for the construction of a 2-ft. 8-in. by 4-ft. concrete sewer from Addington Street to Beatrice Road.

MIDDLETON (LANCS.).—*Concrete Tubes.*—The T.C. has accepted the tender of Messrs. Ellis & Sons, Leicester, for the supply of reinforced concrete tubes.

PORTSMOUTH.—*Concrete Shelter, etc.*—The T.C. has accepted the tender of Mr. J. Croad, Portsea, at £8,100, for the construction of a reinforced concrete store, shelter, etc.

SELBY.—*Concrete Bridge.*—The West Riding C.C. has accepted the tender of the Yorkshire Hennebique Contracting Co., Ltd., at £21,860 8*s.* 11*d.*, for the construction of a reinforced concrete bridge over the canal at Selby.

SHEFFIELD.—*Concrete Houses.* — The Corporation has accepted the tender of Messrs. Henry Boot & Sons, Ltd., for the erection of 1,000 concrete houses.

ST. HELENS, LANCS.—*Kerbs.*—The B.C. has accepted the following tenders for the supply of concrete kerbs :—S. H. Bonsall, Cheadle Heath, 2,000 yds.; Premier Artificial Stone Co., Chester, 1,500 yds.; Shap Granite Co., Shap, 2,000 yds.

STAFFORD.—*Culverts.*—The Corporation has accepted the tender of Messrs. G. Sandy & Co., Stafford, at £876, for the construction of two concrete culverts, and alterations to the road leading to the sewage farm.

TILBURY. — *Concrete Houses.* — The U.D.C. has accepted the tender of the Triangular Construction Co., Ltd., at £19,565, for the erection of 43 concrete houses.

TILBURY.—*Fencing.*—The U.D.C. has accepted the tender of Mr. R. V. Simcock, Marlborough Grove, Rhyl, at £462 0*s.* 8*d.*, for supplying and fixing about 1½ miles of reinforced concrete fencing.

TREGANON. — *Concrete Bridge.* — The D.C. has accepted the tender of Messrs. Hobrough & Co., Ltd., Gloucester, at £5,989, for the construction of a reinforced concrete bridge at Trecefel.

WEYMOUTH.—*Concrete Jetty.* — The T.C. has accepted the tender of Messrs. K. Holst & Co., 1 Victoria Street, London, at £2,733, for the construction of a reinforced concrete jetty on the Weymouth side of the harbour.

WOLVERHAMPTON. — *Concrete Houses.*—The T.C. has accepted the tender of Mr M. A. Boswell for the construction of 1,000 concrete houses.

WOLVERHAMPTON. — *Concrete Wall.* — The T.C. has accepted the tender of Messrs. Wilson, Lovatt & Sons for the construction of a reinforced concrete wall at Commercial Road Station.

WOLVERHAMPTON. — *Foundations.* — The T.C. has accepted the tender of Messrs. Wilson Lovatt & Son, at £396 17*s.* 8*d.*, for concrete foundation work for a rotary water screening plant.

WORCESTER. — *Concrete Walls.* — The T.C. has accepted the tender of the Reinforced Concrete Construction Co., Manchester, at £1,692 8*s.* 3*d.*, for the construction of two reinforced concrete walls at the sewage works.

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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EDITORIAL NOTES.

Speed in Concrete Construction.

RAPID-HARDENING Portland and aluminous cements are comparatively new to the constructional world, and a little time will be needed before builders, contractors and road-makers have adapted their organisation and methods to take full advantage of the facilities and economy these new cements offer. Recent observations go to show that some users of such cements have not advanced the organisation of work to reap the full benefit of their rapid-hardening properties, and it must be remembered that the additional price paid for rapid-hardening cement for a road foundation, for instance, is wasted if the road surface is not applied and the road put into use in less than a week, for if a longer period is allowed an ordinary cement would serve the purpose equally well. This is not an argument in favour of using ordinary cement for road work, because there are few cases in which rapidity of road laying or repair is not an overwhelming advantage and the

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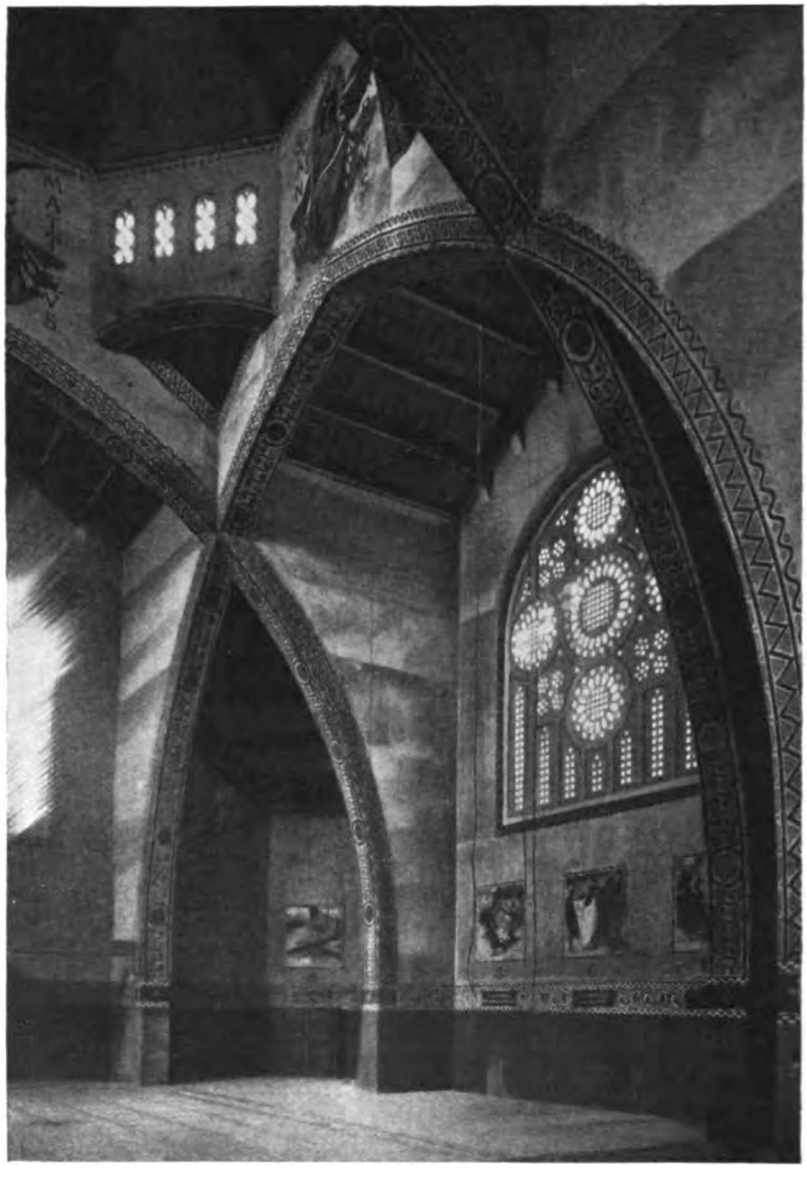

[MM. Droz & Marrast, Architects.

Church of St. Louis, Vincennes : Interior View.

Reinforced Concrete Church in France. (*See p.* 193.)

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extra cost of a special cement is of small moment compared with the direct and indirect savings accruing from reducing the period during which a highway is wholly or partly out of use. The highest economy where rapidity of concrete construction is desirable—and such cases are becoming more numerous every day—lies in the use of rapid-hardening Portland or aluminous cement combined with an organisation that permits of shuttering being removed and re-used, or concrete being built upon, or roads being opened to traffic, in periods commensurate with the hardening properties of the particular cement used.

The application of aluminous cement commenced in France some years before this country, and more experience has been gained in that country in arranging the speed of work to take advantage of the rapid-hardening properties of the cement. In the streets of Paris we have seen aluminous-cement concrete road foundations laid in the morning, opened to foot traffic in the afternoon, covered with wood blocks the next morning, and available for vehicular traffic a few hours later. It is when work is done on such a system as this that concrete work is actually cheapened by the use of a high-priced cement, and progress in this direction must come from the cement users. Rapid-hardening cements are frequently of greater strength than ordinary cements at all ages, and there is thus an additional scope for economy by taking advantage of this quality in the design of the mixture and carrying out the work. In this connection it is for the engineers to take the steps that will lead to economy.

Fresco Painting on Concrete.

PAINTING in true fresco is one of the most durable forms of wall decoration known, as is proved by the fresco paintings dating from about 1500 B.C. discovered recently by Sir Arthur Evans at Knossos; it is, however, an art little used nowadays, probably owing to the preference of the mural decorator to work under studio conditions rather than accept the arduous alternative which true fresco painting demands of working on the wall face itself, often on scaffolding. In true fresco painting the pigment is held by the setting of the lime to which it is applied, and as no adhesive is added to the pigment it is essential that the painting be carried out while the plaster is wet, that is, on the same day. It is this kind of fresco which has been used to decorate the walls of the reinforced concrete church of St. Louis, at Vincennes, illustrated in this issue, and this application of the most permanent type of decoration to the most permanent materials of construction is a point of considerable interest to the architect. Speaking at a meeting of the Royal Institute of British Architects last month, Mr. Frederick R. Hiorns, F.R.I.B.A., referred to this church as "an example of a newer and more truthful tendency in architecture, which recognised not only constructive truth and the relation of form to formation, but also the corporate spirit in building that admitted the work of painters, sculptors, and others in its execution and resultant expression." With frank acceptance of a material which will enter still more into all phases of architecture as time goes on, and an honest endeavour to get the best artistic results from a material of proved fitness from a constructional point of view, concrete offers an opportunity for the mother of the arts again to gather to her side the sculptor and the painter, all working in the one material.

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Bridge Design.

No special pleading is required on behalf of reinforced concrete as a material for the construction of bridges. In the great majority of cases it is the cheapest material for the purpose, and in addition there is the strong point in its favour—especially at the present time—that it provides much more work for local labour than any other type of bridge.

Although the practical advantages of reinforced concrete for bridge construction are now generally realised, the opinion is still sometimes expressed that the material lacks the power of endowing a structure with dignity and beauty. This impression has probably been brought about owing to the advocates of reinforced concrete laying emphasis on its cheapness rather than on its architectural possibilities, but whatever the cause such an opinion is hardly possible in face of the results. A structure in concrete, as in any other material, is beautiful or not according to the skill of the designer, and a reinforced concrete bridge can be as good to look at as one in any other material and the cheapest at the same time. No new material of construction has ever been used to the best artistic advantage in the early days of its use ; practical requirements are the first consideration, and it is only after many years of use, after designers have become fully aware of the possibilities and limitations of a material, that architectural styles are evolved and masterpieces produced.

We therefore welcome, in the comparatively early days of concrete bridges, the appeal of the Minister of Transport that the greatest attention should be given to the design of the large number of new bridges to be built in the near future. Of all structures a bridge is built to last, and it would be a matter of regret for generations to come if the countryside and cities were disfigured by ugly bridges. As the Minister of Transport says, although stability and economy are the chief objects in view at the present time, there is no reason why beauty should not be reached through these considerations alone. Too often a building with excellent proportions is spoilt by the application of ornament which is not only meaningless and unnecessary but also expensive. We can point to many beautiful examples of reinforced concrete bridges which owe their attractiveness solely to the outline naturally reached in designing the material to meet the requirements.

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REINFORCED CONCRETE COOLING TOWERS.

New Type Developed in Holland.

In a paper read recently before the Society of Engineers, Mr. H. A. Rickwood, A.M.I.E.E., described a new type of reinforced concrete cooling tower developed by Professor F. K. van Iterson, Director of the Dutch State Mines at Heerlen, and Mr. G. Kuypers, Chief Building Surveyor of the City of Amsterdam.

The lecturer said that having studied and experimented upon cooling towers of various designs from the best Belgian, Dutch, French and German and British makers, Professor van Iterson became convinced that the best results could be obtained economically only when much larger towers were used than formerly with collecting tanks of great capacity. Being convinced that a long rectangular shape did not permit the water to be distributed in a thoroughly uniform manner, through longitudinal troughs, secondary gutters, etc., he and his collaborator came to the conclusion that the most satisfactory arrangement would be to provide large circular towers, about 100 ft. in diameter, and to arrange that the inlet pipe should deliver in the centre of the tower, the water being carried toward the outer parts by means of radiating troughs, feeding, in their turn, gutters arranged in parallel concentric lines, each triangular sector being thereby fed in exactly the same way.

There were difficulties to overcome. First, it was desirable that the internal cooling stack of the shell should be unconnected with the shell, and also that the interior of the shell should be free from traverse stays and bracings which would interfere with the regularity of the air draught owing to the eddies their presence provoked, thus disturbing the efficiency of the stack; but the construction of a structure 100 ft. or so in diameter and some 120 ft. high without

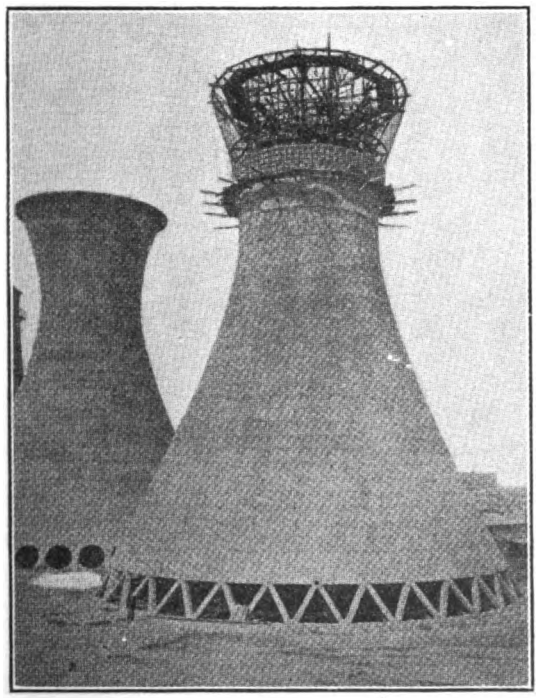

Fig. 1.

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any such stays was no easy structural problem, especially if it were desirable that the weight of the structure should be kept within reasonable limits so as to avoid the necessity of expensive foundations.

It was found that the best solution would be to use ferro-concrete for the construction of the shell, because it was monolithic, rigid, and light. With a view to reducing the shearing stresses in the shell, the principles which were first applied by the late Gustave Eiffel for the construction of his tower in Paris were adopted, and a double curved surface was used, the shell being in fact an hyperboloid (*Fig.* 2). This arrangement, excellent from a structural point of view, was found to regulate the air draught, as the air streams become automatically directed toward the vertical axis of the tower. The hyperbolic shape produced a good draught, and the widening part of the shell above the neck, while adding greater strength, had the further effect of considerably reducing the speed of the air column, saturated as it was in that part of the shell with aqueous vapour, converting air velocity into pressure.

Fig. 2 gave the vertical and horizontal

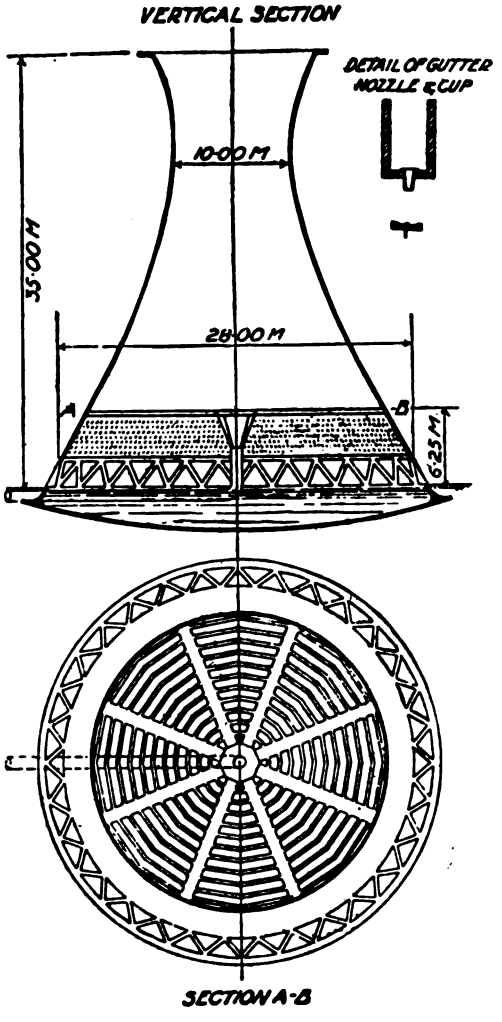

Fig. 2.

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section of a tower built in accordance with this general principle. The great lightness of the shell was evident from the vertical section. The inlet water pipe was brought to the centre of the tower at a level which coincided practically with that of the water in the pond. It then bent up and discharged through an invert conical shape funnel (thus reducing the speed of the water at the inlet). At the top of the cooling stack eight radiating troughs carried the water towards the shell, and in their turn fed eleven subsidiary gutters. Along the length of the latter spray nozzles were distributed, which discharged the water on earthenware saucers fixed some 18 in. lower down. The shape of these was such that the splashing of water was transformed into a fine spray. The drops of water fall upon wooden laths, and then from lath to lath down into the pond.

Temperatures were given of the hot air above the inlet level, recorded on six thermometers distributed equally along the diameter of the tower, two examples being given—one from a tower erected at Pit Emma and the other at Pit Wilhelmina, at Heerlen. These figures were found to be very even, there being no great difference between the temperature in the middle of the tower and close to the shell.

Reinforced Concrete Cooling Towers at Liverpool

The success of this new type of cooling tower in Holland has recently led to the adoption of a similar method of construction at the Liverpool Corporation electricity works, where three reinforced concrete towers are now being built on the Mouchel-Hennebique system as the first part of a scheme including the construction of twelve cooling towers. *Fig.* 6 is a vertical section through one of the towers at the Liverpool electricity works. Each tower is 122 ft. 6 in. high measured from the bottom of the cooling pond beneath it, and 130 ft. high from foundation level, the diameter being 100 ft. at ground level and 107 ft. 5 in. at foundation level. The lightness of the shell is evident from the section, the wall being 12 in. thick at the base, progressively reduced to 7½ in. and 5 in. in the lower half, the upper half being only 4½ in. thick. The lower part of the tower encloses a cooling pond with a water depth of 5 ft. 6 in., and connected with this are discharge ducts with sumps and manholes, the ducts leading to a general suction culvert. Reinforced concrete supports for the inlet main, of 28 in. internal diameter, and for the timber cooling stacks, are shown in *Fig.* 6, where only part of the stacks is included.

The inlet pipe is brought to the centre of the tower, where it bends up and discharges through a funnel inverted conical shape (thus reducing the speed of the water at the point of discharge). At the top of the cooling stack eight radiating troughs carry the water towards the shell, and in their turn feed eleven subsidiary gutters. Along the length of the latter are distributed spray nozzles which discharge the water on earthenware saucers fixed some 18 in. lower down. The shape of these is such that the splashing water is transformed into a fine spray; the drops of water fall from lath to lath down into the pond.

The design aims at balancing resistance to air flow and the provision of a maximum cooling surface. By comparing a tower of 100 ft. diameter with four towers each of 50 ft. diameter, it will be realised that the cooling surface in both cases is practically the same, but that the circumference of the shell of the large tower is only half the aggregate circumferential length of the four smaller towers. The resistance to the air flow due to friction against the shell is thereby largely reduced, and the cost of the shell is also reduced.

Each of the towers now under construction at Liverpool was designed to cool 467,000 gallons of water an hour from 102·5 deg. to 83 deg. Fahr., the wet bulb temperature of the air being 55 deg. The three towers will provide for cooling nearly 1½ million gallons an hour, and the ultimate completion of the entire set for over 5½ million gallons an hour.

The Liverpool installation is being carried out to the designs and under the direction of the City Electrical Engineer, Mr. H. Dickinson, M.Inst.C.E., with the assistance of Mr. P. J. Robinson, M.I.E.E.

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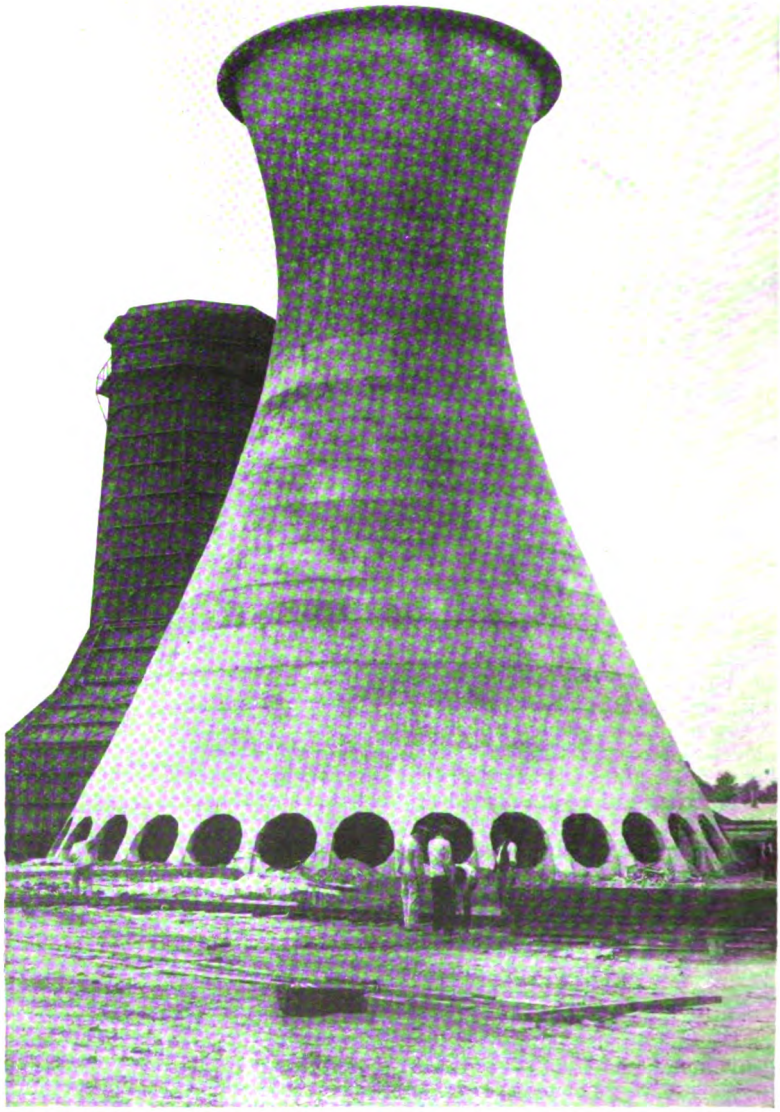

Fig. 3.—One of the First of the New Type to be Built in Holland.

Reinforced Concrete Cooling Towers.

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Fig. 4.—General View of Group of Towers.

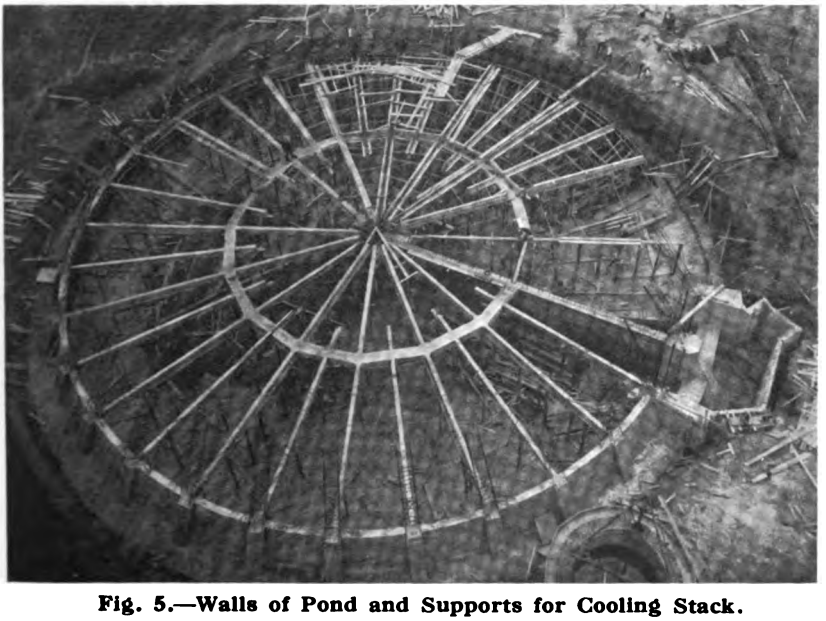

Fig. 5.—Walls of Pond and Supports for Cooling Stack.

Reinforced Concrete Cooling Towers at Liverpool.

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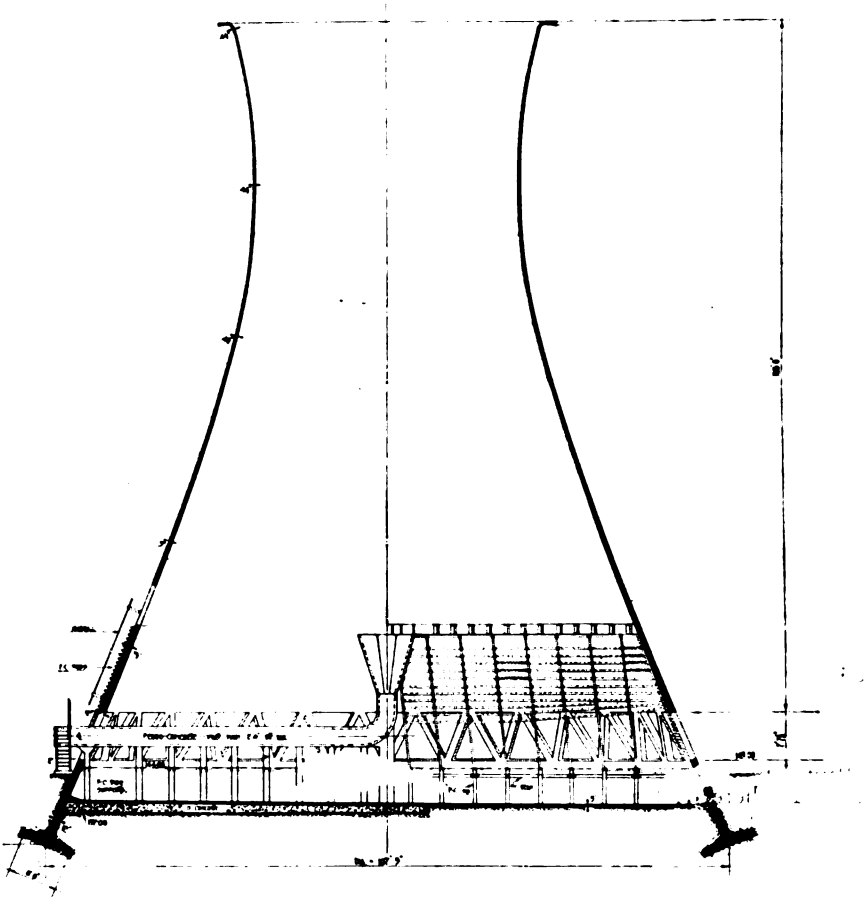

Fig. 6.—Typical Section of New Reinforced Concrete Cooling Towers at Liverpool.

the Deputy City Electrical Engineer. Details of the reinforced-concrete construction were prepared by Messrs. L. G. Mouchel & Partners, Ltd., and the contractors are Messrs. Joshua Henshaw & Sons, of Liverpool.

International Roads Congress.

THE fifth International Roads Congress is to be held in Milan in September, 1926, on the invitation of the Italian Government, and an international exhibition of materials and methods of road construction will be held in Milan at the same time. The papers to be read will deal with all types of road construction and materials, and also with such subjects as special roads for motor traffic, development and planning of towns in the interests of traffic, financial arrangements, etc. A special committee, called the Road Congress British Organizing Committee, has been set up at the Ministry of Transport, 7, Whitehall Gardens, London, S.W.1, to advise and assist those who propose to visit the Congress from this country.

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SAFETY OF RETAINING WALLS AGAINST SLIDING.

By M. REINER, D.Sc. Eng. (Vienna).

(1) To calculate the safety of a retaining wall against sliding, the usual procedure is as follows :—

Let P be the horizontal pressure,
W the vertical weight,
ϕ the coefficient of friction,

then the resistance due to friction is assumed to be $F = \phi \times W$, and the factor of safety against sliding $n = \frac{F}{P} = \phi \times \frac{W}{P}$

That means: Should the coefficient of friction for certain reasons, as, e.g. wet surface, etc., be only $\frac{\phi}{n}$ instead of ϕ, the retaining wall will just stand the horizontal pressure without sliding, but any further diminishing of this coefficient would be disastrous for the wall.

(2) This method of calculation is generally incorrect. It is correct only in the special case where the point of application of the resultant force of P and W is in the centre of the base, i.e. if the stresses on the base are uniformly distributed. In every other case it is misleading, as will presently be shown.

(3) First let us remember that the frictional resistance is not a definite force like W or P. It is a *resistance*, which means that it will vary with the applied force which it resists. F is only the *limit* up to which the frictional resistance is able to increase. If P is smaller than F, then the actual resistance (let us call it F^1) will be not F but P. If P is greater than F, then the body (in our case the retaining wall) will start to slide and no [illegible] i.e. friction of rest, as we do not [illegible] friction of motion) will take place at all.

(4) How does the actual frictional resistance F^1 ($F^1 \leqq F$) come into play? Like every resistance it comes into play when one body (in our case the retaining wall) acts on another (the soil), as the reaction of the soil against the retaining wall. The action produces a min[illegible] elastic deformation of the second body, and this minute elastic deformation develops elastic stresses, which at the surface of contact appear as frictional resistance.

(5) The distribution of F^1 over the surface of contact depends therefore entirely on the elastic properties of the two bodies acting upon each other. F^1 may be distributed uniformly or in some other way, but this distribution has no connection with the distribution of F. The variations of F^1 will conform to the variations of the deformation as every stress to the strain.

(6) The distribution of F, on the other hand, can very easily be determined. As F is equal to $\phi \times W$ and ϕ is a constant, the distribution of F will be exactly the same as that of the weight (normal pressure) W.

(7) Let us take as an example the case of a table resting on two plates (instead of legs) * and carrying a weight W (see *Fig.* 1), which may be applied at the distance a from the left support (1), leaving a distance b from the right support (2).

* Plates have been taken here instead of legs to simplify the problem by reducing it to two dimensions.

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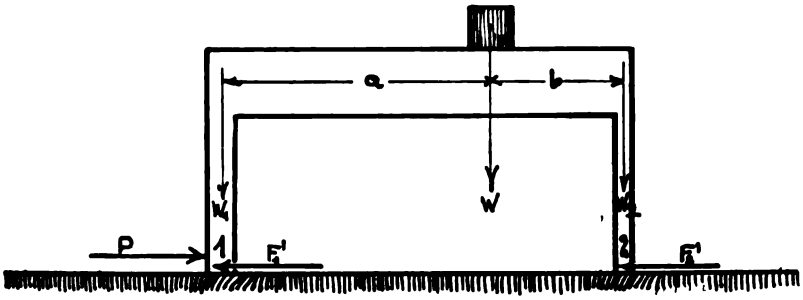

FIG. 1.

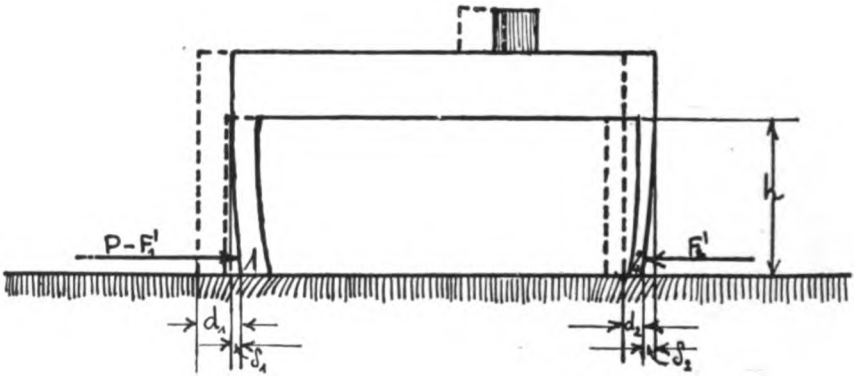

FIG. 2.

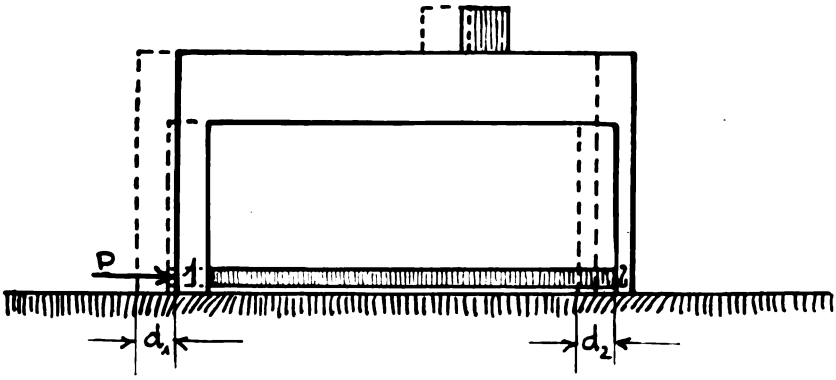

FIG. 3.
SAFETY OF RETAINING WALLS AGAINST SLIDING.

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The weight W may be resolved into two component forces:

$$W_1 = W\frac{b}{a+b}\ ;\ W_2 = W\frac{a}{a+b} \quad . \quad . \quad . \quad . \quad (1)$$

and on these forces the ultimate frictional resistances will depend as follows:

$$F_1 = \phi W_1 = \phi W\frac{C}{a+C};\ F_2 = \phi W_2 = \phi W\frac{a}{a+C} \quad . \quad . \quad (2)$$

F_1 and F_2 cannot be surpassed by the actual frictional resistances at the two supports, i.e. :—

$$\left.\begin{aligned} F_1^1 \leqq F_1 \leqq \phi W\frac{b}{a+b} \\ F_2^1 \leqq F_2 \leqq \phi W\frac{a}{a+b} \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

(8) Now, if a horizontal force P tends to move the table as indicated (see *Fig.* 1), then the plate (1) will slightly move in the direction of P by a displacement d_1; this will cause a deformation of the floor at the surface of contact (supposing no sliding occurs); this deformation will develop stresses in the floor and these stresses will act as a resistance to the support (1), i.e. as the frictional resistance F_1^1. But if (1) moves towards the right it will cause moving also of support (2) towards the right by a displacement d_2 $(o \leqq d_2 \geqq d_1)$, which will similarly result in a frictional resistance F_2^1, acting at support (2).

(9) The question is now to calculate F_1^1 and F_2^1 from the elastic properties of the floor and the table.

With regard to the floor, the stresses in the floor will evidently be proportional to the displacements d_1 and d_2, which gives

$$F_1^1 = ad_1\ ;\ F_2^1 = ad_2 \quad . \quad . \quad . \quad . \quad (4)$$

where a is a constant proportionate to the modulus of elasticity of the material of which the floor is composed.

(10) With regard to the table, we have first the static equation

$$F_1^1 + F_2^1 = P \quad . \quad . \quad . \quad . \quad . \quad (5)$$

which expresses that the table will remain in equilibrium.

Equation (5) can also be written thus :—

$$P - F_1^1 = F_2^1 \quad . \quad . \quad . \quad . \quad . \quad (5a)$$

and under these two forces, $P - F_1^1$ and F_2^1, the table will act as an elastic frame and undergo a deformation as shown in *Fig.* 2.

If we assume that the plate of the table is of much greater stiffness than the legs, then the legs will act as cantilevers and δ_1 and δ_2 may be calculated as the deflections of the beams, thus giving

$$\left.\begin{aligned} \delta_1 = \frac{P - F_1^1}{EI_1}\frac{h^3}{3} = \frac{P - F_1^1}{\beta I_1} \\ \delta_2 = \frac{F_2^1}{EI_2}\frac{h^3}{3} = \frac{F_2^1}{\beta I_2} \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad (6)$$

where β is a constant proportionate to the modulus of elasticity of the material of which the table is composed.

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Furthermore, it is clear from *Fig.* 2 that

$$\delta_1 + \delta_2 = d_1 - d_2 \quad . \quad . \quad . \quad . \quad . \quad (7)$$

(11) We have now six equations (4), (5), (6) and (7) with six unknowns F_1^1, F_2^1, d_1, d_2, δ_1, δ_2. From these we get easily:

$$\left.\begin{aligned} F_1^1 &= P\,\frac{\dfrac{1}{\alpha} + \dfrac{1}{\beta}\left(\dfrac{1}{I_1} + \dfrac{1}{I_2}\right)}{\dfrac{2}{\alpha} + \dfrac{1}{\beta}\left(\dfrac{1}{I_1} + \dfrac{1}{I_2}\right)} \\ F_2^1 &= P\,\frac{\dfrac{1}{\alpha}}{\dfrac{2}{\alpha} + \dfrac{1}{\beta}\left(\dfrac{1}{I_1} + \dfrac{1}{I_2}\right)} \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad (8)$$

and this solves the problem if α and β are known.

(12) Two extreme cases can be distinguished. First, the case when the material of which the floor is composed is of much greater rigidity than the material of the table; and, second, the case when it is reversed.

To consider the first case we have to put $\alpha = \infty$ or

$$\frac{1}{\alpha} = o \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

and this gives

$$\left.\begin{aligned} F_1^1 &= P \\ F_2^1 &= o \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad (10)$$

That means: the whole of the friction acts at support (1) and no friction at all acts at support (2).

To decide whether the table will slide or not, the expressions of equation (10) have to be compared with the terms of equation (3). Then it will be seen that the ultimate value P can reach, if the table should not slide, depends on the position of the weight W on the table.

It is:

$$\lim P = \phi \times W \quad \text{if } a = o,$$

$$\lim P = \frac{\phi W}{2} \quad \text{if } a = b,$$

$$\lim P = o \quad \text{if } b = o$$

(13) To consider the second case we have to put $\beta = \infty$ or

$$\frac{1}{\beta} = o \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

and this gives

$$F_1^1 = F_2^1 = \frac{P}{2} \quad . \quad . \quad . \quad . \quad . \quad (12)$$

That means: the friction is uniformly distributed on both supports.

This case can easily be demonstrated as a table with braced legs, assuming that the brace is incompressible (take, for example, a steel brace between wooden

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legs). The result would be that the displacements d_1 and d_2 would always be equal (see *Fig.* 3) and that means: $F_1^1 = F_2^1$.

Now let us take, e.g., $P = \phi W$ . . . . . (13)

then

$$F_1^1 = F_2^1 = \frac{\phi W}{2} \quad . \quad . \quad . \quad . \quad . \quad (14)$$

If therefore the weight W is applied in the centre of the span, i.e. $a = b$, then the table will remain at rest, but if, for example, $a = 2b$, then we have from equation (3)

$$\left.\begin{array}{l} F_1^1 \leqq \frac{1}{3}\phi W \\ F_2^1 \leqq \frac{2}{3}\phi W \end{array}\right\} \quad . \quad . \quad . \quad . \quad . \quad (15)$$

which means that support (1) will first come into sliding, and that therefore F_1^1 will be diminished to zero; then the whole force P will act at support (2), which cannot resist more than two-thirds of P, and therefore support (2) will also come into sliding: i.e. the table will slide altogether.

(14) Now let us take instead of a braced table a solid block. Then all displacements at different points of the base can be taken as being equal, so that the actual frictional resistance is uniformly distributed.

$$F_1 = \frac{P}{d} \quad . \quad . \quad . \quad (16) \quad \text{(see } Fig. \text{ 4)}$$

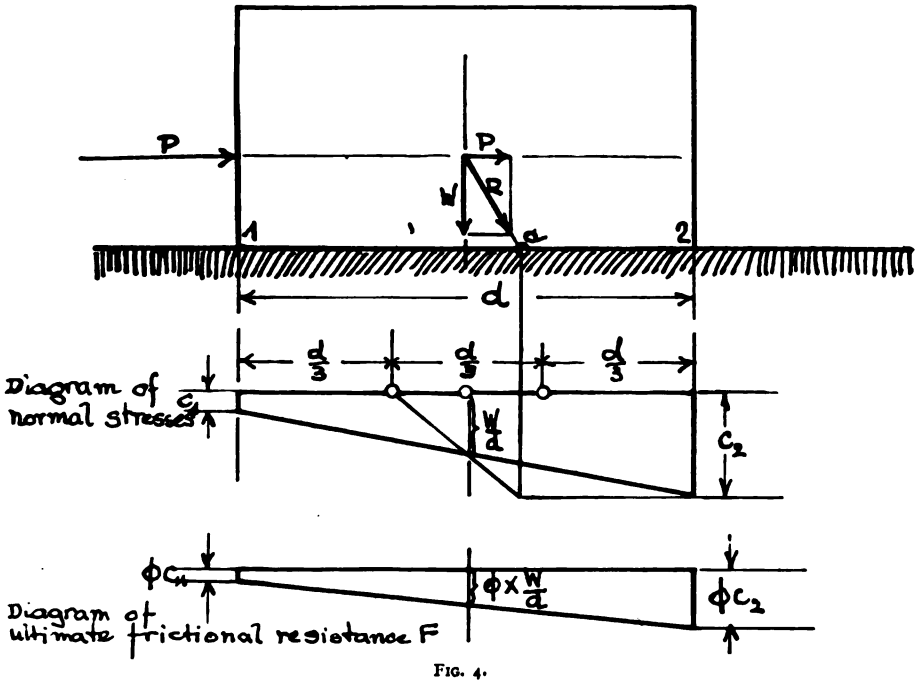

FIG. 4.

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The distribution of the normal pressure at the base will generally be trapezoidal. The distribution of the ultimate frictional resistance will thence also be trapezoidal, as explained in para. 6.

Let us take, for example,

$$P = \phi \times W \quad . \quad . \quad . \quad . \quad . \quad (17)$$

Then it follows from (16)

$$F^1 = \phi \frac{W}{d} \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

and the block will therefore be safe against sliding if the point of application of the resultant force R is in the centre of the base. But if the point of application "a" is eccentric, as in our case, then F^1 surpasses at the edge (1) the ultimate value at this point $F = \phi \times c_1$ $\left(\text{note that } c_1 \text{ is smaller than } \frac{W}{d} \text{ and therefore } \phi c_1 < \phi \frac{W}{d}\right)$ and the block will come into sliding at this edge.

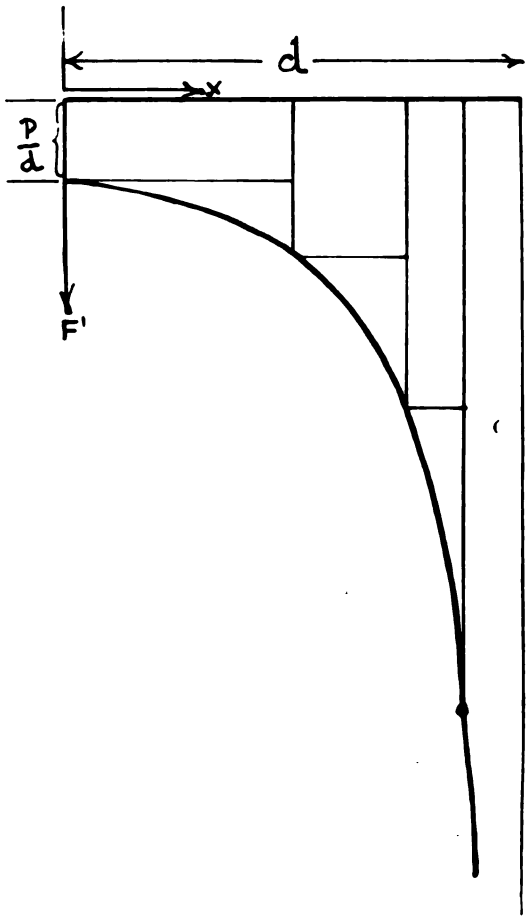

FIG. 5

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(15) The question then is, what will happen after sliding starts at this edge? Will stability be reached, or will the block ultimately come into sliding altogether?

If sliding at edge (1) starts on a length of say x, then the width of the base with regard to frictional resistance has been reduced by this length x and is now $d - x$. Instead of expression (16) we have the following:

$$F^1 = \frac{P}{d - x} \quad . \quad . \quad . \quad . \quad . \quad (19)$$

The curve for F^1 can easily be constructed if we note that

$$F^1_{x=0} = \frac{P}{d}; \; F^1_{x=d} = \infty; \; F^1_{x=nd} = \frac{1}{1-} \; F^1_{x=0}; \; \frac{dF^1}{dx} = -Plg(d-x)$$

By comparison of this curve with the diagram for F (see *Fig.* 4) three cases can be distinguished (see *Fig.* 6).

In case 1 stability will exist from the beginning. In case 2 sliding will proceed up to a length s, but then stability will be reached. In case 3 no stability is possible and the block will slide altogether.

(16) These considerations may be applied to a retaining wall resting on soil if the material of which the retaining wall is composed is of comparatively greater rigidity than the latter. This is often the case, especially if the retaining wall is of reinforced concrete.

(17) To take a definite example, such as occurred to me when checking a calculation submitted to the P.W.D., Palestine, for approval (see *Fig.* 7). The

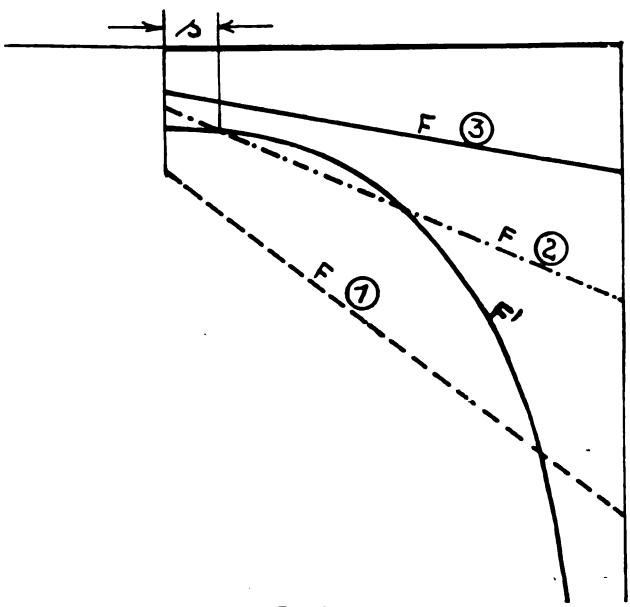

Fig. 6.

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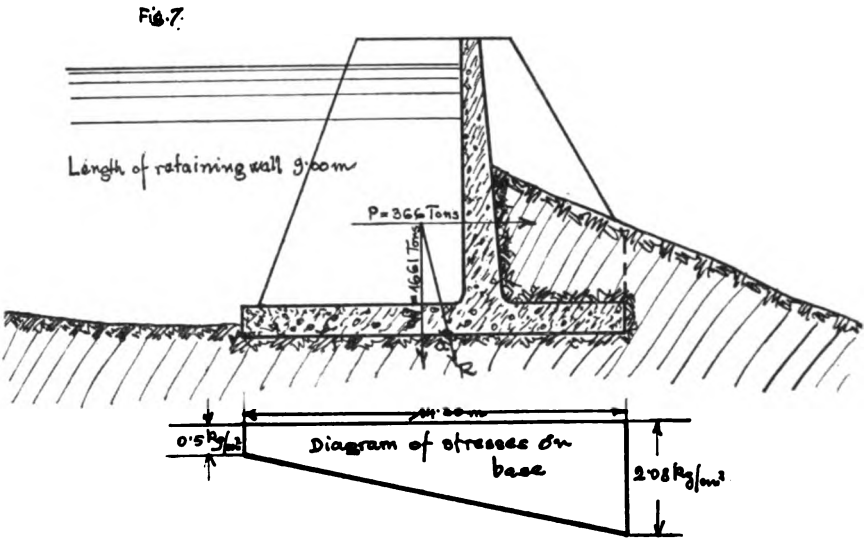

FIG 7

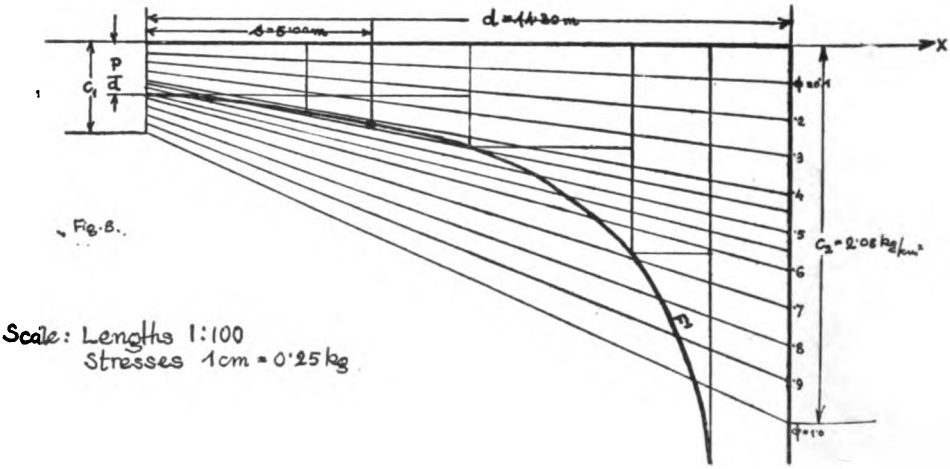

FIG. 8.

SAFETY OF RETAINING WALLS AGAINST SLIDING.

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calculation ran like this: "Taking coefficient of friction 0·5. Force resisting sliding = 830 tons. ∴ Factor of Safety against sliding on base = 2·26."

(18) According to the method of calculation described above, the calculation should be:

Coefficient of friction, 0·5; Factor of safety asked for, 2·26;

$$\therefore \phi = \frac{0{\cdot}5}{2{\cdot}26} = 0{\cdot}22.$$

$$\frac{P}{d} = \frac{366000}{1430 \times 900} = 0{\cdot}28 \text{ kg/cm.}^2$$

After study of *Fig.* 8 it will be found that ϕ should be at least 0·42 to maintain stability. In this case the dam will slide at the inner edge at a length of 5·00 m., but will then reach stability. The factor of safety in this case would be $n = \frac{0{\cdot}50}{0{\cdot}42} = 1{\cdot}19$, which appears to be too low.

If no sliding should occur at all at the inner edge, ϕ would have to be at least $\frac{0{\cdot}28}{0{\cdot}50} = 0{\cdot}56$.

The structure appears therefore not to be safe against sliding, although the designer had, according to the usual way of calculation, good reasons to assume that it would be safe. It has been found advisable to re-design the dam by widening the base, so that the point of application would fall in its centre.

(19) To calculate the safety of a retaining wall against sliding, the procedure should therefore be as follows:

Calculate the vertical weight W, horizontal pressure P, point of application a, minimum stress c_1.

Calculate $\frac{P}{d}$.

Divide $\frac{P}{d}$ by c_1. The result is the necessary coefficient of friction to prevent sliding.

If ϕ is the coefficient of friction to be expected, then

$$n = \phi \div \frac{P}{dc_1}$$

is the factor of safety against sliding.

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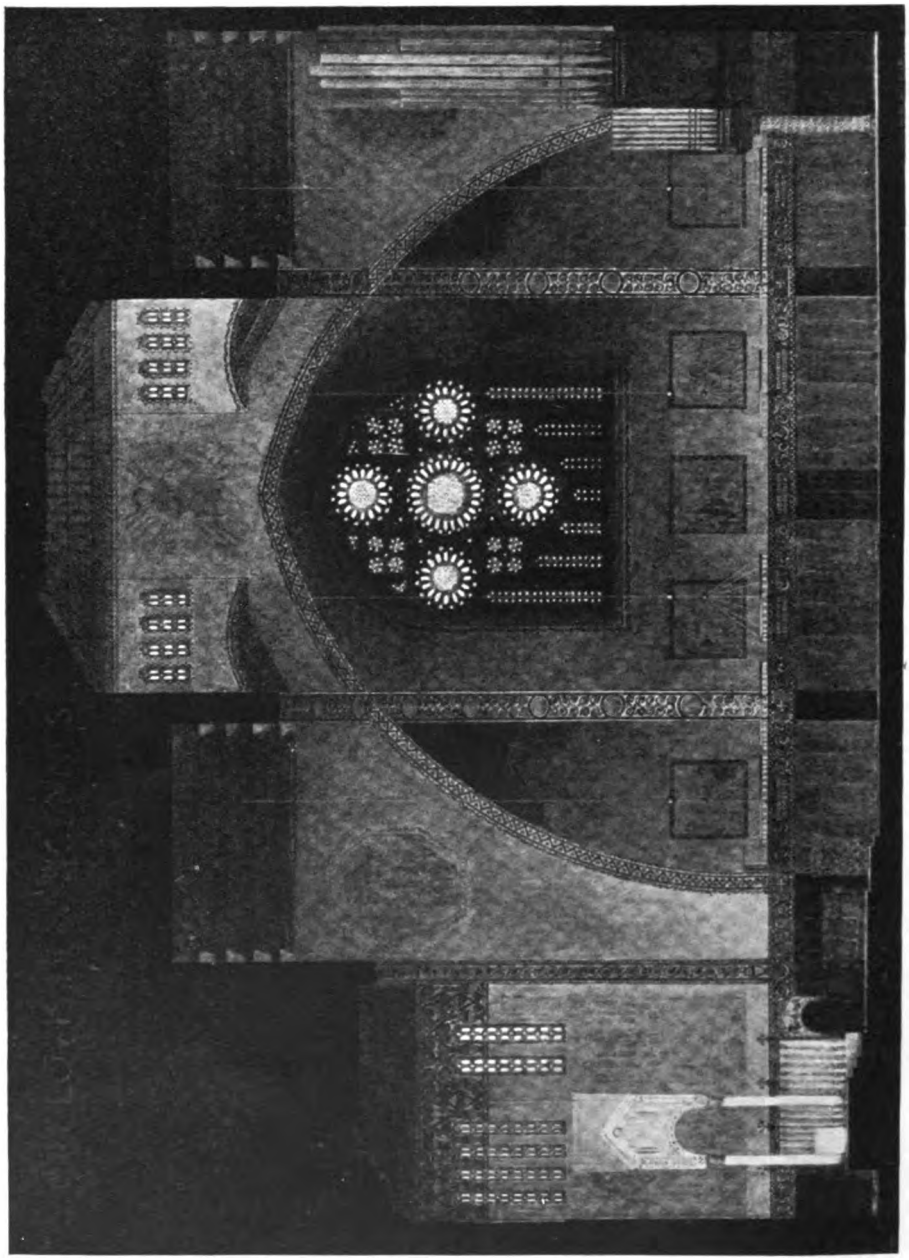

Church of St. Louis, Vincennes: Longitudinal Section. [M. Droz & Marrast, Architects.

(See p. 193.)

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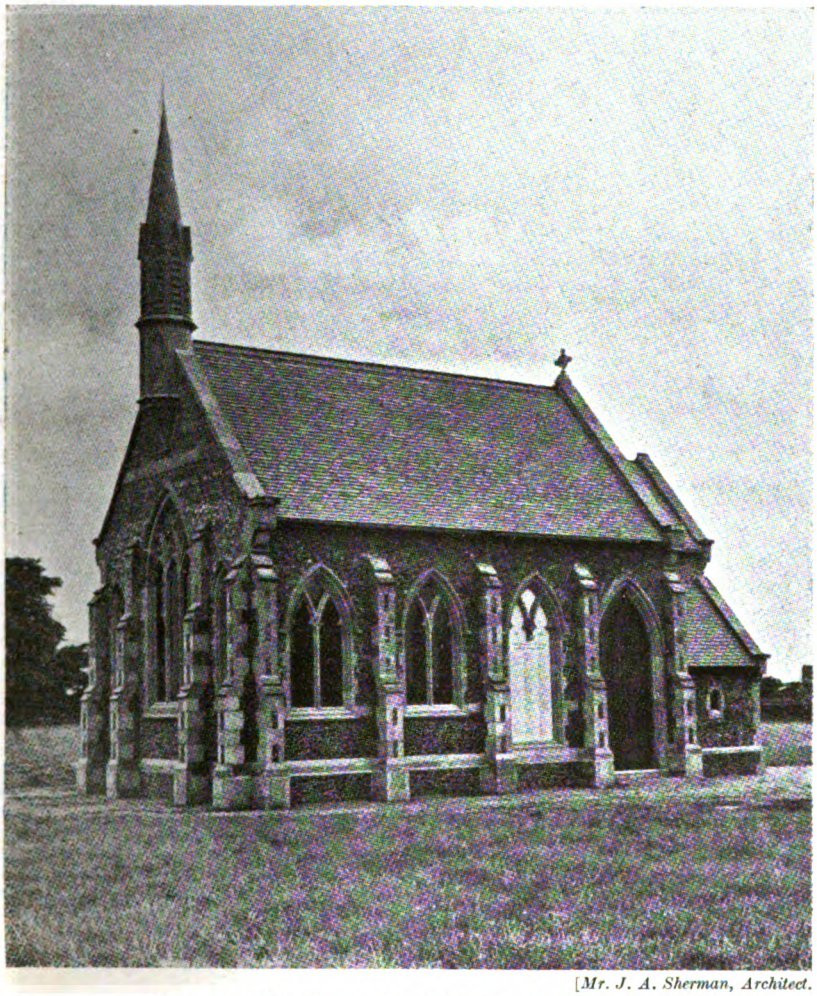
Concrete in Church Architecture.

Church of St. Louis, Vincennes.

Fresco Painting on Concrete.

COMMENCED during the war, but held up during the campaign, the Church of St. Louis, at Vincennes, near Paris, has just been completed. The exterior walls are built in stone and brick, while the roof is supported by four reinforced concrete arches with a clear span of 73 ft. 6 in. As will be seen from the illustrations, the four arches are built in pairs intersecting at right angles, the distance between each pair being 40 ft. On the arches rest the walls of an octagonal skylight. The windows are set in reinforced concrete frames.

A specially interesting feature of this church is the wall painting, which is in parts done in true fresco direct on a fresh lime coating applied to the concrete surface. All the painted decoration of the edges of the arches and on the arch panels and spandrels was applied in this way, and fresco painting was also used to decorate the wall around the door on the exterior wall. The altar screen, choir, pulpit and other features are decorated partly with ceramic tiles and partly with coloured cements.



[*Mr. J. A. Sherman, Architect.*

Leiston War Memorial Chapel. (*See p.* 195.)

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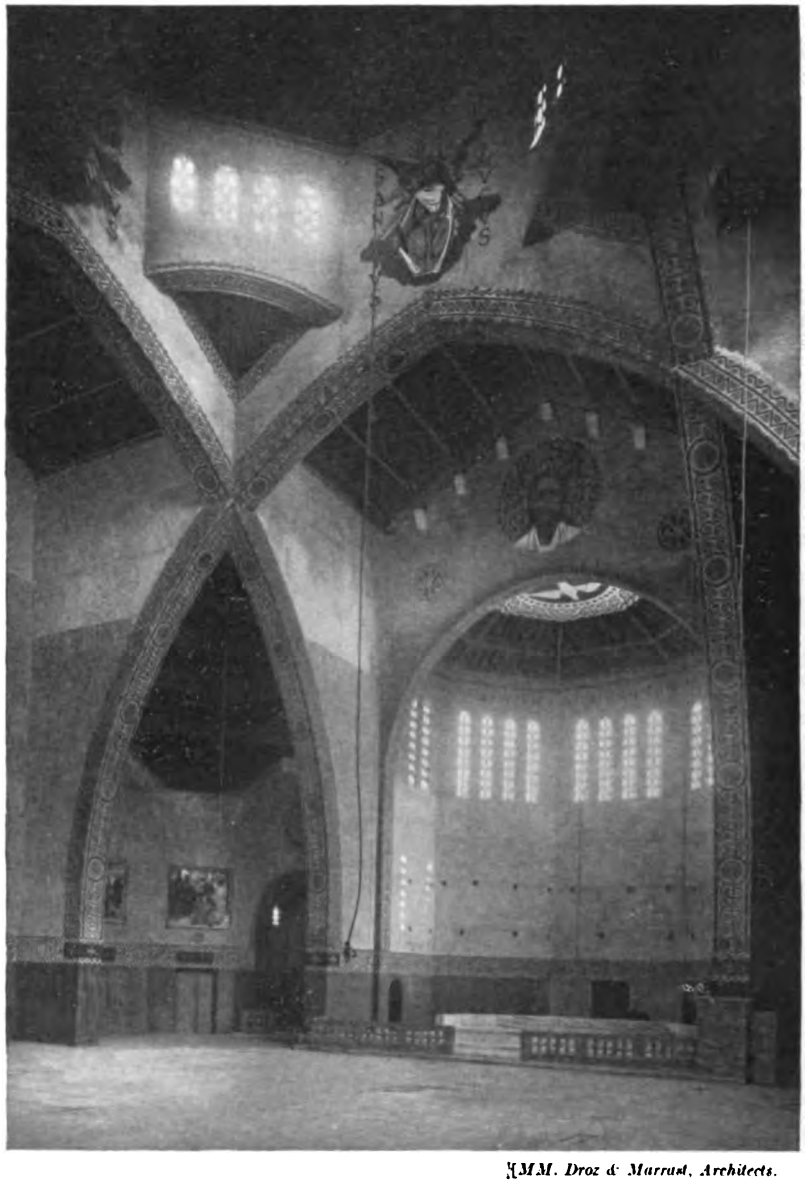

[MM. Droz & Marrast, Architects.

Church of St. Louis, Vincennes: Interior View.

Reinforced Concrete Church in France. (*See p.* 193.)

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A Concrete Chapel.

THE chapel illustrated on page 193 has just been built as a war memorial in the cemetery at Leiston, Suffolk. It is in the Early English style of architecture, and consists of a nave, small chapel, and vestry. The walls are built hollow. The outer wall is a 6-in. concrete wall faced with "knapped" flints (black) and the inside is of 4½-in. concrete blocks. The plinths, buttresses, window sills, arches, mullions and copings are of granolithic stone made in moulds and cast on the site, the materials used being granite chippings, shingle, sand and cement. The roof is open inside and covered externally with sand-faced flat tiles of a rich red colour. There is a copper bell turret at the west end. The most prominent position in the chapel facing the roadway was selected for the memorial tablet of white marble, on which are inscribed the 122 names of the fallen. The total cost of this chapel was £1,230.

The architect is Mr. J. A. Sherman, architect and surveyor, of Ipswich. The work was carried out by Messrs. Smyth Bros., builders and contractors of Leiston, and the whole of the work was done locally. We are indebted to Messrs. W. Brown & Co., Ipswich, Ltd., builders' merchants, of Greyfriars Road, Ipswich, for our illustrations.

[Mr. J. A. Sherman, Architect.

Leiston War Memorial Chapel: The Commemorative Tablet.

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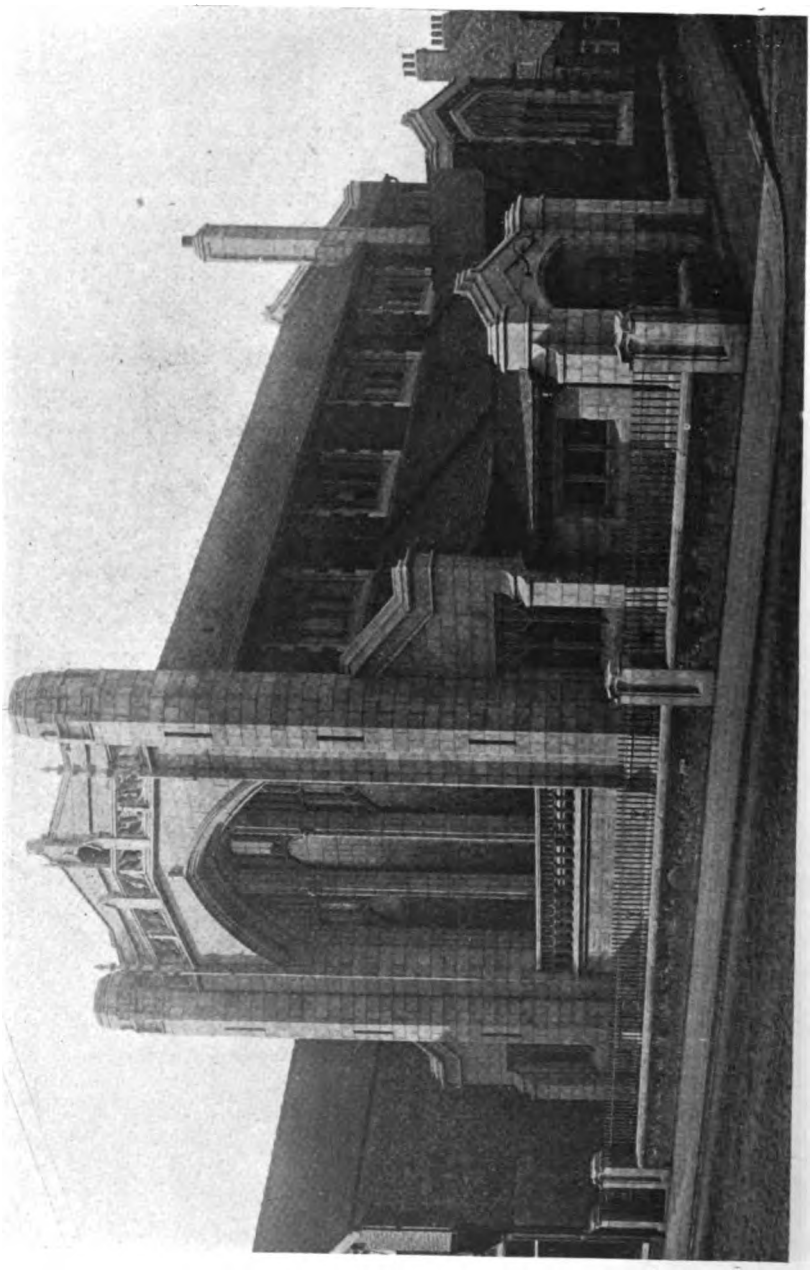

Park Presbyterian Church, Middlesbrough. *Messrs. Kitching & Archibald, Architects.* (*See p.* 197.)

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Park Presbyterian Church, Middlebrough.

CAST concrete stone has for many years been extensively used for the façades of commercial buildings, and owing to the high cost of natural stone is being made increasing use of for this purpose by many of our leading architects. Particularly is this the case in London, where a large proportion of the important new commercial buildings have cast-stone frontages.

It is not often, however, that cast stone has been used for ecclesiastical buildings, and the two churches illustrated in this issue are of particular interest in showing what can be done in this direction, in the production of a pleasing building with no loss of dignity and with a very considerable saving of cost.

In Park Presbyterian Church, Middlesbrough, which occupies a site at the corner of Linthorpe Road and Clive Road, cast stone made with white cement has been used for the east front, north porch, tracery windows to aisles and clerestory, gable copings, chimney shaft, boundary walls and the pillars on the exterior and interior arches and steps to chancel. The whole of this cast-stone work, which was made with white and "Pelican" brand cements supplied by Messrs. G. and T. Earle, Ltd., of Hull, was made by the

[*Messrs. Kitching & Archibald, Architects.*

Park Presbyterian Church, Middlesbrough.

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Hull Concrete Stone Co., Ltd., of Stoneferry, Hull, and forms a pleasing contrast with the red brick of which the walls are built.

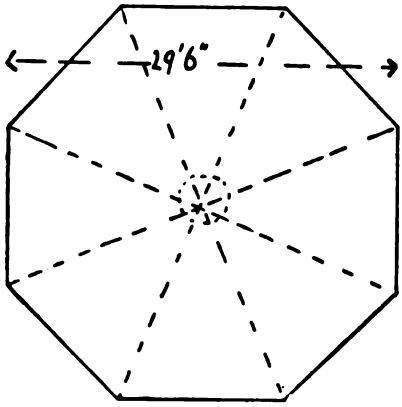
The architects are Messrs. Kitching and Archibald, of Middlesbrough, and the builders Messrs. Stephen Coates, Ltd., of Ormesby, Middlesbrough.

Questions and Answers relating to Reinforced Concrete.

Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.—ED.

Floor Beams.

QUESTION.—The accompanying sketch shows the plan of an octagonal room, the floor above which is to be supported by four reinforced concrete beams intersecting at a central boss 6 ft. in diameter.



Each beam is to be 24 in. deep by 12 in. wide, reinforced with seven 1-in. rods. The rods are to be in two layers of 4 and 3. I am in doubt as to how to arrange the steel at the central intersecting point, and should be much obliged if you could show me how it can best be done.

F. C. B.

ANSWER.—It is not possible to arrange the steel in a satisfactory manner at the central intersecting point with the sizes and number of rods given, as some of the tension rods would be situated on the top surface even when placed directly on top of each other across the full width of the beam. Unless the floor is carrying an exceptional load it would appear that the steel specified is excessive; the difficulty that now presents itself is due to some error in the design, and in any case the ratio of steel to concrete is not satisfactory. A complete check cannot be made because the floor load, thickness of slab—if any—and other essential information is lacking, but it is suggested that the particulars and calculations should be checked over by the querist before any solution of the problem is attempted. If the loads and stresses are found to be correct, it will be necessary to design the floor on the principle of one, or at most two, main beams which will span right across the octagon, and the remainder of the beams can be designed as secondary beams which are supported by the main beam at the point of intersection. If uniformity of size is necessary, the secondary beams can be designed with the same sectional area of concrete but with a reduction in the steel, and the tensional rods can be hooked over the rods in the main beam to avoid the complication that arises at the crossing; in addition, some of the rods in the secondary beams can be cranked up to come into the upper surface. It is not possible to give any further assistance in this matter unless the whole of the necessary particulars are given.

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Modern Methods of Construction.

A PAPER entitled "Applications in Building and Foundations of Modern Engineering Construction" was read by Dr. Oscar Faber, O.B.E., D.Sc., M.Inst.C.E., at a recent meeting of the Royal Institute of British Architects. The paper included the following:

Theory.

Soils in nature which we sometimes have to found on or retain may be said to lie somewhere between the following four extreme cases:—

(*a*) Those possessing neither cohesion nor friction.

(*b*) Those possessing friction but no cohesion.

(*c*) Those possessing cohesion but no friction.

(*d*) Those possessing cohesion and friction.

The first of these (*a*) are liquids. For stability of a foundation for vertical loads the depth of the foundation below the free surface must be

$$h = \frac{p}{w} \quad . \quad . \quad . \quad . \quad (1)$$

Where h = depth of foundation below free surface in ft., p = pressure on foundation in lbs. per sq. ft., w = weight of soil *above* foundation in lbs. per cu. ft. In other words, the pressure on the foundation must equal the pressure in the soil at this depth due to the weight of earth above it.

$$p = wh \quad . \quad . \quad . \quad . \quad (2)$$

This pressure will be referred to as the hydrostatic pressure of the soil at this depth. Values are given in Table I.

TABLE I.

HYDROSTATIC PRESSURE AT VARIOUS DEPTHS IN SOIL WEIGHING 120 LBS. PER CU. FT. $p = wh$.

Depth in ft.	Pressure in Lbs. per sq. ft.	Pressure in Tons per sq. ft.
5	600	·27
10	1,200	·54
15	1,800	·80
20	2,400	1·07
25	3,000	1·34
30	3,600	1·61
35	4,200	1·88
40	4,800	2·15

If the foundation is at a less depth than given by 1 then the foundation will sink, and if it is at a greater depth it will rise. It must be exactly at that depth, neither more nor less.

It may be considered a stretch of imagination to consider a pure liquid as the limiting condition of a soil, but the properties of a liquid are approximated to by soft mud which is sometimes found on the banks of rivers which have passed through clay deposits. The Thames is a notable case in point, and the mud on many of the marshes between Greenhithe and Sittingbourne approximates closely to this condition below the top surface, sometimes for a depth of 25 ft. or so.

Pressures against retaining walls built in front of this material approximate closely to the pressures which would exist if the walls were retaining liquid, and embankments built on these marshes have to be given an extremely flat slope if they are to be free from risk of failure, and even then they settle continuously. The river wall along these marshes is constantly settling and men are constantly employed in raising it.

Soils possessing friction but no cohesion (*b*), were considered in great detail by Rankine. These conditions are practically fulfilled in the case of loose dry sand, which is a perfectly reliable foundation provided it is free from risk of erosion or having its conditions disturbed in other ways, and providing the depth of the foundation is arranged at a proper level.

Rankine showed that for stability the depth of the foundation below the free surface must be

$$h = \frac{p}{w}\left(\frac{1 - \sin\theta}{1 + \sin\theta}\right)^2 \quad . \quad . \quad . \quad (3)$$

where h, p and w have the same significance as before.

θ is suggested by Rankine to be the angle of repose of the material; that is to say, the greatest slope to the horizontal at which the material will stand. Subsequent experiments have shown, however, that the pressures calculated from this formula agree much better with experience if θ is taken as the angle whose

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tangent gives the coefficient of friction of the material; that is, the relation between the force required to produce lateral motion across a horizontal section of the material to the vertical force across that section. The formula in (3) gives the least depth at which such a foundation is stable, and if a foundation is placed at a higher level it will sink down to this depth. Any depth below this level will also be a stable foundation until the maximum permissible depth is reached, when h has the value in equation 4,

$$h = \frac{p}{w}\left(\frac{1 + \sin\theta}{1 - \sin\theta}\right)^2 \quad . \quad . \quad . \quad (4)$$

If this is exceeded the foundation would then tend to rise, as the upward pressure would be greater than the downward weight.

As compared with a liquid, the permissible depth, instead of having the definite figure of $h = \frac{p}{w}$, may now lie anywhere within a certain range, partly above this depth and partly below it, and in proportion as the depth differs from $h = \frac{p}{w}$ we rely upon the friction of the material to secure stability.

Rankine understood quite well that these formulæ were only properly applicable to soils which approximated to his assumptions that there should be no cohesion, and that the particles were prevented from moving only by friction. Nevertheless his formulæ have since that date been somewhat indiscriminately applied to soils which do not approximate to these assumptions, such, for example, as clay. Clay is a material with a quite definite and considerable cohesion, as is quite clear to a navvy attempting to cut it with a spade, who has to exert considerable force in shearing through the material. It is clear from an examination of formula (3) that a soil of this character will carry no load if placed on the surface, because if h is 0 clearly p must be 0 also.

This confirms experience when dealing with dry sand, as anyone who has walked across a really dry sandy beach will remember. θ* for clean sand has values varying from about 65° for the dry material, gradually increasing to about 69° as the percentage of water increases to about 21, and then gradually reducing to about 62° when the sand is saturated with water. This increase in θ is no doubt due to the surface tension between the particles when it is moist, which gives it something approaching cohesion.* This again is within common experience, as anyone who has tried to make sand castles at the seaside will remember, as damp sand can be made to stand with a vertical face, whereas very dry or very wet sand both refuse to do so.

It is not advisable for the height of a foundation to differ from its most stable position $h = \frac{p}{w}$ to the full extent of the expressions represented by equations (3) or (4), because there would in that case be no factor of safety, and a foundation should be lower than the upper limit or higher than the lower limit by an amount depending on the factor of safety desired.

Rankine may perhaps be slightly to blame for the misuse of his formulæ to soils to which they do not properly apply, because although he recognised that most actual soils, except pure sand, have cohesion, he considered that such cohesion is liable to be broken down by the action of water and frost, and that therefore the only thing which could be relied upon for permanent stability was the friction. This, of course, is not so, since a foundation on blue clay, for example, at a considerable depth is sufficiently protected against the action of both frost and water to retain indefinitely any cohesion which it may possess. If Rankine's formula were really strictly applicable to ordinary soils other than sand, or soils approximating thereto, then the safe pressures would always be given in terms of the depth in accordance with equation (3).

Table II gives the safe pressures on soils of different depths calculated in accordance with equation (3), taking w, the weight of the material, as 120 lbs. per cubic ft., and θ, which I will call the friction angle, as 30° and 60° respectively.

* See the paper on "Experiments on Earth Pressures," by P. M. Crosthwaite. Institution of Civil Engineers, 1916.

* It would therefore be more accurately dealt with as a cohesion, in the manner suggested for case (*d*).

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TABLE II.

MAXIMUM CARRYING CAPACITY * OF LOOSE GRANULAR MATERIAL WITHOUT COHESION AT VARIOUS DEPTHS FROM RANKINE'S FORMULA:

$$p = wh\left(\frac{1 + \sin\theta}{1 - \sin\theta}\right)^2$$

taking w = 120 lbs./cu. ft.

	Carrying capacity with $\theta = 30°$.		Carrying capacity with $\theta = 60°$.	
Depth in ft.	Lbs. per sq. ft.	Tons per sq. ft.	Lbs. per sq. ft.	Tons per sq. ft.
0	0	0	0	0
5	5,400	2·41	116,400	52
10	10,800	4·82	232,800	104
15	16,200	7·23	349,200	156
20	21,600	9·64	465,600	208
25	27,000	12·05	582,000	260
30	32,400	14·46	698,400	312
35	37,800	16·87	814,800	364
40	43,200	19·28	931,200	416

As previously stated, these pressures would not be safe as they would contain no factor of safety, but even if they were halved or divided by three they will be seen to produce figures which startle one. In particular, the tremendous increase with increasing value of θ is startling. In the author's view the higher pressures indicated at the great depths would not ordinarily be suitable on building foundations, as they would be accompanied with a considerable settlement owing to the consolidation of the particles. He knows of no case where foundations on sand or material without cohesion has exceeded 12 tons per sq. ft. at any depth.

It must be remembered that loose sand —to which alone these formulæ strictly apply—has voids between the solid particles varying from 40 to 20 per cent. or less, depending on the grading of the particles. It is therefore clear that considerable movement may take place owing to this consolidation which is ignored in the Rankine formulæ. Rankine assumed the material is incompressible, but at high pressures it would compress considerably. Consider a material like sand making only point contact between its particles. It is clear that if there is a pressure of, say, 10 tons per sq. ft. on the foundation, then the pressure of contact between the particles would be more than 10 tons per sq. ft. in proportion as the area of contact is less than the area of the foundation. Taking this ratio to be only one-third, the stress on the material would be at least 30 tons per sq. ft. The author would consider 10 tons per sq. ft. on a material like sand an upper limit irrespective of Rankine's figures. Higher pressures would have an insufficient factor of safety against fracture of the sand particles and would produce settlement.

The L.C.C. Regulations give a maximum of 2 tons per sq. ft. on brown clay and 4 tons on blue clay or ballast. No mention is made of the depth of these materials, so that on the L.C.C. basis the stability would not appear to depend on the depth at all. In the author's view this is overstating the facts in the other direction, and the author would consider it at least permissible, with suitable safeguards, to allow an increase of pressure of $p = wh$ (given in Table I) over and above what the same soil is capable of resisting near the surface; in other words, to add the hydrostatic pressure to its carrying capacity near the surface. This is absolutely conservative and makes no additional demand upon either friction or cohesion over and above what is already resisted by the material at the selected pressure at small depths. In other words, if a certain ballast or clay were found to be capable of resisting 2 tons per sq. ft., say at a depth of 4 ft. (the depth should, of course, always be below the frost line or the line above which the clay is affected by seasonal variations of moisture contents), then the same soil at a depth of 30 ft. would be capable of carrying an additional 1½ tons per sq. ft. without making any greater demands on the material either by way of friction or of cohesion.

An examination of Table I will show that these figures are quite important. In many cases it would even be safe to exceed them, but they at least are in the author's view conservative. The depth must, of course, be measured from a surface which exists at the time of building and can reasonably be relied upon to remain for the lifetime of the building. It is clear, for example, that if a building be founded on a material approximating to Rankine's assumptions, viz., loose sand, and a pressure were adopted depending on a certain height of material above the foundation, and this were subsequently

* These figures must be reduced by a suitable factor of safety for working pressures.

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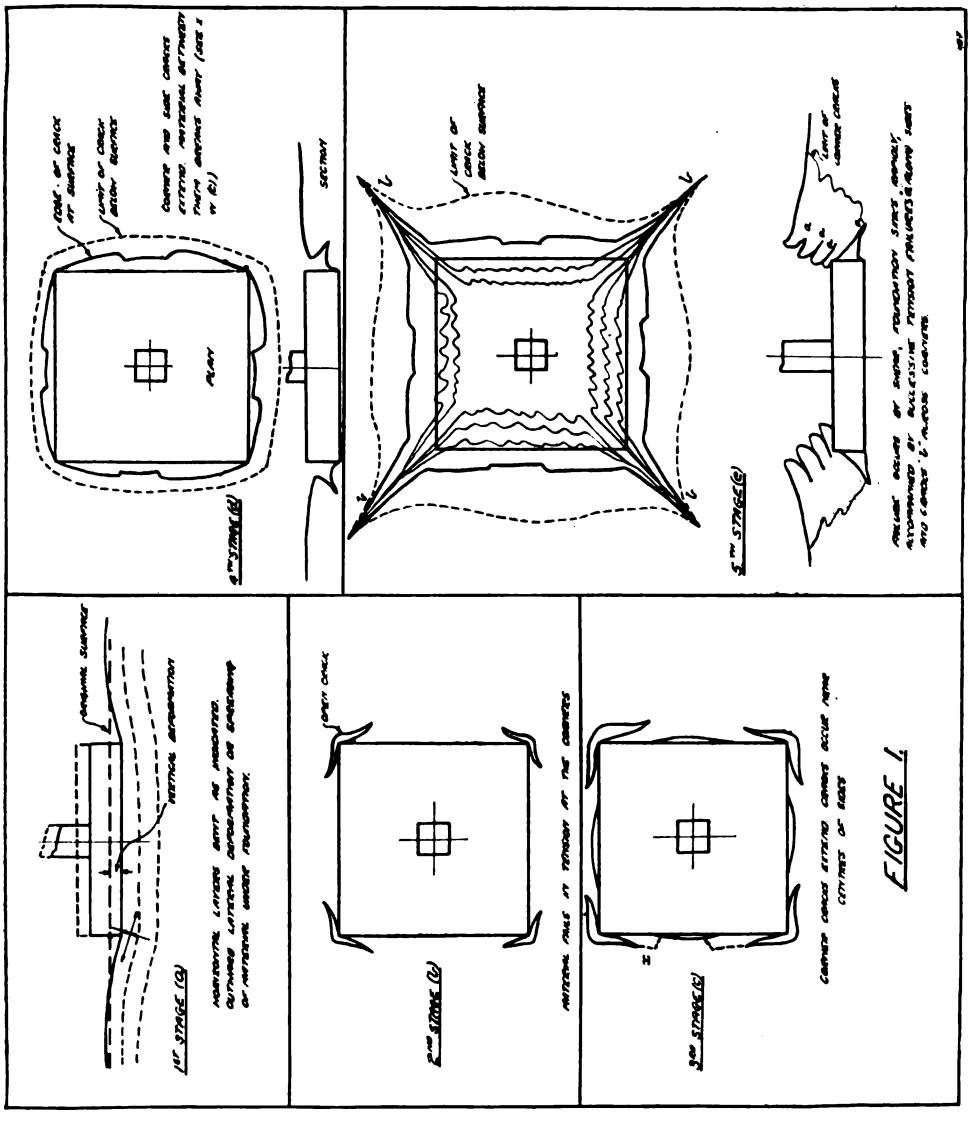

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excavated away for an adjacent building or for any other reason, the foundations of the first building would be jeopardised even if the foundations of the second building went no deeper and possibly not so deep as those of the first building. Fortunately we do not in practice often deal with materials like sand, and therefore the point is not so important as it otherwise would be, but it is interesting to remember that for such a material the load is in this sense carried by the material above the foundation just as much as by the material below it.

Enough has perhaps been said to show that Rankine's formulæ lead to results which, though useful and interesting, and nearly true for loose sand, are widely discordant with experience on common soils. The common practice of applying them to all soils is therefore ridiculous and should cease.

Coming now to class (*c*) soils possessing cohesion but no friction. It is curious that no study appears to have been made of these, yet they are instructive and interesting. A soft clay with considerable water contents approximates much more closely to this case than to a material obeying Rankine's formulæ with a low value of θ.

The author has therefore made an experimental study of this class of material, which, not being complete, he will only refer to in a general way. The nearest approach that he knows of an actual material to the condition of the ideal material possessing cohesion but no friction is jelly. The fact of cohesion in this material is evinced in the fact that loads may be placed on it without settlement (though not without slight deformations which are common to all solids under stress), and by the fact that it will stand up with a vertical face. The comparative absence of friction is evinced by the way vibrations in the material continue undamped, and it is well known that damping of vibration is a measure of friction. Jelly as an experimental material has the great advantage that cohesion can be varied (by varying the gelatine contents) and what happens below the surface is readily seen.

Fig. 1 illustrates what happens when a square foundation resting on such a material is loaded with a gradually increasing load till failure occurs. The figure needs little explanation. It is remarkable that the increase of deformation with increasing load is almost constant through all the stages 1 to 4. When stage 5 is reached the foundation punches its way to the bottom. Failure occurs when the horizontal layers, which are clearly lengthened under deformation, are stretched more than they will stand. This naturally occurs first at the corners where the lengthening is greatest.

The failure seems to suggest two things.

(1) That as final failure occurs by a kind of shear or punching, the breaking load might vary with the periphery rather than with the area. In other words, that if foundation A is 5 ft. sq. and B is 10 ft. sq., the breaking load on B might be nearer twice that on A rather than four times. My experiments so far indicate that it lies somewhere between the two, but more experiment is required and is under way.

(2) That foundations rounded in plan and having the bottom edge rounded might carry higher loads. This also needs more experiment, and this is in hand.

A mathematical analysis indicates that a material having cohesion but negligible friction will have a carrying capacity of

$$p = wh + 4k \quad . \quad . \quad . \quad (5)$$

where k is the shear resistance of the material. The shear strength of the clay in any particular foundation can be measured by experiment. Values given by Mr. A. L. Bell, M.Inst.C.E.,* are as follows :—

	Tons per sq. ft.
Very soft puddle clay . .	0·2
Soft puddle clay . . .	0·3
Moderately firm clay . .	0·5
Stiff clay	0·7
Very stiff boulder clay . .	1·6

This table is difficult to apply because the description is insufficient, but tests can be made on the actual material of a foundation.

Taking the values of 0·07 for stiff clay to which he refers and ignoring any friction, the carrying capacity at various depths would work out as in Table III.

* Paper before Inst. Civil Engineers, 1914.

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TABLE III.

CARRYING CAPACITY AT VARIOUS DEPTHS OF SOIL HAVING SHEAR STRENGTH OF 0·7 TON PER SQ. FT. AND NEGLIGIBLE FRICTION. $p = wh + 4k$.

Depth in ft.	Carrying capacity. Lbs. per sq. ft.	Tons per sq. ft.
0	6,270	2·8
5	6,870	3·07
10	7,470	3·34
15	8,070	3·60
20	8,670	3·87
25	9,270	4·14
30	9,870	4·41
35	10,470	4·68
40	11,070	4·95

In his experiments, Mr. Bell found that stiff clay had some friction, which has the effect of increasing the carrying capacity with increasing depth more than is indicated by Table III.

It will be seen that these figures agree much more closely with experience with common soils than those in Table II, based on Rankine's formula, showing again that the assumption of cohesion without friction is a closer approximation to fact for common soils, and particularly clay, than Rankine's assumption of friction without cohesion.

Soils possessing cohesion and friction (*e*). This classification, with suitable values for the cohesion and friction, is applicable to all soils. Mathematical analysis shows that the maximum pressure on a foundation at a depth h is given by

$$p = wh \tan^4\left(\frac{\pi}{4} + \frac{\theta}{2}\right) + 2k \tan^3\left(\frac{\pi}{4} + \frac{\theta}{2}\right)$$

$$= 2k \tan\left(\frac{\pi}{4} + \frac{\theta}{2}\right) \quad . \quad . \quad . \quad (6)$$

where k is the shear strength of the soil and θ is the angle whose tangent is the coefficient of friction of the material. Both these can be measured and tested on an actual soil. All the preceding formulæ can be derived from this.

Putting $k = 0$, and $\theta = 0$, it reduces to

$$p = wh \quad . \quad . \quad . \quad . \quad (2)$$

Putting $k = 0$, it reduces to

$$p = wh\left(\frac{1 + \sin\theta}{1 - \sin\theta}\right)^2 \qquad (3) \quad (\textit{slightly rearranged})$$

$$\left[\text{since } \frac{1 + \sin\theta}{1 - \sin\theta} = \tan^2\left(\frac{\pi}{2} + \frac{\theta}{2}\right)\right]$$

Putting $\theta = 0$, it reduces to

$$p = wh + 4k \quad . \quad . \quad . \quad . \quad (5)$$

our formula for cohesion without friction.

Table IV shows the result of applying this formula to a soil having a shear strength $k = 0·7$ ton per sq. ft. and a friction angle $\theta = 7°$, which Mr. Bell gives for a particular "stiff clay."

TABLE IV.

CARRYING CAPACITY AT VARIOUS DEPTHS OF SOIL HAVING A SHEAR STRENGTH OF 0·7 TON PER SQ. FT. AND A FRICTION ANGLE OF 7°. (See formula 6.)

Depth in ft.	Carrying capacity. Lbs. per sq. ft.	Tons per sq. ft.
0	8,060	3·6
5	9,040	4·04
10	10,020	4·48
15	11,000	4·91
20	11,960	5·34
25	12,950	5·78
30	13,930	6·23
35	14,910	6·66
40	15,900	7·1

It will be seen that the results divided by a factor of safety of 1·5 would agree well with experience of London blue clay.

The formula is, however, equally applicable to ballast and other soils by using the appropriate constants, which can be ascertained by testing for any actual material.

Formulæ 5 and 6 avoid a difficulty which many designers must have felt, which may be expressed as follows: A certain foundation carrying, for example, 200 tons is desired to be founded at a certain level on a foundation 10 ft. sq., giving 2 tons per sq. ft. At this level, however, the soil is not considered good enough to carry 2 tons per sq. ft. and the hole has then to be carried down an additional 20 ft. in depth before a satisfactory soil is reached. At the lower depth the foundation, however, now has to carry not only the original 200 tons but also the weight of the material forming the pier between the higher level originally contemplated and the lower level. This may form a considerable proportion of the 200 tons to be carried, and therefore the pressure per sq. ft. at the lower level will necessarily be greater than what it was at the higher level.

A strict interpretation of the Building

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Act as it stands at present would not allow of this increase of pressure, yet it is quite clear that had the foundations been founded at the higher level then the soil at the lower level would in point of fact have carried it plus the 20 ft. of earth intervening; and, therefore, it is quite reasonable to take the carrying capacity of the soil at the lower level as 2 tons per sq. ft. plus the weight of the soil between the two levels. This is exactly what formula 5 does.

(*To be continued.*)

Book Reviews.

History of the Portland Cement Industry in the United States. By R. W. Lesly, J. B. Lober, and G. S. Bartlett.

Chicago: International Trade Press, Inc. Price $3½.

In a profusely illustrated volume of more than 300 pages the authors have collected and arranged in a manner suitable for the layman as well as the technical man a history of the progress of the Portland cement industry in the United States. Mr. Lesly was associated with the pioneers of the industry in America some fifty years ago, and is thus in an exceptional position to write on the material which is making possible the concrete age in the United States. In a foreword, Mr. Floyd W. Parsons says: "The cement industry of the United States has established for itself a position of such permanence and importance in America's industrial life that neither arguments nor eulogies are needed to gain public recognition and appreciation for the great business. We may search the whole field of material virtues and yet find nothing that so commands our respect as does permanence. Cement means concrete; concrete means stone; and stone spells eternity, so far as our finite minds can comprehend. The development of methods to manufacture great quantities of synthetic stone has given our present civilisation the durability it so much needed. The creation of a great cement industry has removed in large degree the serious menace to life of a coming timber famine. The rapid progress of science and engineering in our country has been made possible by our cement mills, which have given us the essential material on which to found our prosperity."

The volume is divided into convenient chapters dealing with the different phases of the development of the industry, such as the importation of cement into the United States, the gradual growth of the industry in that country and the improvement of manufacturing processes and mechanical methods. Appendices give an account of the history of the Portland Cement Association of the United States, standard specifications and tests, and historical notes on Portland cement manufacturers. The output of Portland cement in the United States is stated to have now reached 150,000,000 barrels a year, whilst twenty-five years ago the production was only 10,000,000 barrels.

Sell's Directory of Registered Telegraphic Addresses.

London: Business Directories, Ltd. Price 45s.

With this, the 1925 edition, "Sell's Directory of Registered Telegraphic Addresses" attains its 40th year of publication. Since 1894 the lists in this Directory have been compiled from official lists supplied by the authority of H.M. Postmaster-General, and the directory has increased in size till it now forms a volume of over 3,000 pages. The telephone numbers of all firms in the lists are given by arrangement with the Post Office, and thus subscribers have in one volume the telephone numbers of the most important firms throughout the United Kingdom. Every name in the official alphabetical lists has a marginal number to facilitate and cheapen the cabling of names and addresses. This edition also contains the number of the postal delivery office as well as the district initials for all addresses in the London postal area. In Section II the abbreviated telegraphic addresses are alphabetically arranged, enabling the addressee to identify the sender of a telegram, even though the message be signed—as is so frequently the case—by a registered code word only. The Classified Trades List contains nearly 130,000 names of firms in the United Kingdom, arranged in order of trade under some 3,500 trade headings.

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Fig. 1.—Before Strengthening.

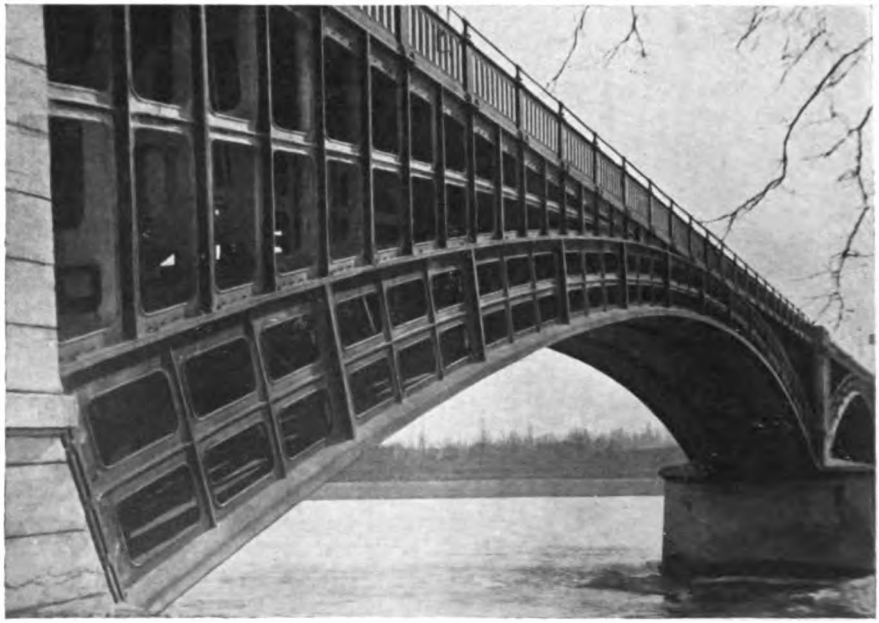

Fig. 2.—After Strengthening.

Strengthening a Cast-Iron Railway Bridge with Reinforced Concrete.
(*See p.* 207.)

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Strengthening a Cast-Iron Railway Bridge with Reinforced Concrete.

An interesting account of the repairs and strengthening of an important railway bridge by means of reinforced concrete is given in a paper in *Les Annales des Ponts et Chaussées*, by M. de Boulongne, Chief Engineer of Roads and Bridges and Chief Engineer for Steel Structures to the Paris, Lyons and Mediterranean Railway. In close proximity to La Voulte the bridge carries the Livron-Privas section of the P.L.M. Railway across the Rhone, and comprises five cast-iron arches, each of 180-ft. span, the total length being about 1,075 ft., including the masonry abutments. Completed in 1861, cracks later developed owing to the great rigidity of the transverse members between adjacent arch ribs and to the oxidisation of iron filings in the mastic cement used for filling the joints, and consequently it became impossible to allow any locomotives but those of the lightest type to cross the bridge.

Before the war a scheme was prepared for replacing the bridge by one of the straight steel girder type, but on the termination of hostilities it was decided to adopt the less costly alternative of repairing and strengthening the existing structure by means of additional steel members and reinforced concrete.

The scheme embodied several novel features. The cast-iron transverse members between the arch ribs, which had become very defective, were replaced by steel frames. *Fig.* 4 is a cross section through the crown of an arch showing the four arch ribs and the connecting cast-iron frames, of which there were only two at this point; *Fig.* 5 is a cross section near the haunch of an arch showing the new steel frames attached to the cast-iron arch ribs.

Before removing the defective cast-iron members the new steel frames provided at each joint between arch voussoirs were fixed in position by bolts passing through the holes originally used for the bolts connecting the voussoirs, so avoiding drilling operations. When the new frames were fixed the corresponding cast-iron members were removed, and special precautions were taken owing to the nature of the mastic cement fixing the ends of the members into the housings cast on the cheeks of the voussoirs.

The strengthening of the lower flange of each arch rib was effected by the addition of steel bars, securely attached to the cast-iron work and afterwards encased in concrete. As shown in *Figs.* 7 and 8, the general strengthening of the lower flange was effected by the addition of two round bars of 36 mm. diameter on each side of each voussoir, to which they were attached by steel brackets spaced 2 m. apart, a few extra bars of the same diameter being placed at points where the flanges were defective. These main bars were tied laterally by bars of 12 mm. diameter, spaced 20 cm. apart and passed through holes in the web of the voussoirs. Tightly wedged in the holes, the ties pass around the 36-mm. bars and connected to the reinforcing bars of the intrados slab shown in the drawings. Consequently the main and tie-bars were already securely attached to the voussoirs before being encased in concrete, and the wedging of the tie-bars in the voussoir holes is a definite safeguard against any tendency of the concrete to slide along the web of the voussoirs. Owing to the magnitude of the spans the main reinforcing bars had to be made up of three lengths for each arch, and the separate lengths were connected by screwed unions and at the springings of the arches the ends of the bars were fixed by bolts in steel gussets in turn bolted to the springing plates of the ribs.

The main bars, attached to the voussoirs and with their ends anchored to the springing plates, are subjected to all compressive and tensile stresses to which the lower flange is at the same time subjected, and therefore give substantial relief to the latter. Moreover, as the elastic modulus of steel is double that of cast-iron the addition of two 36-mm. bars is equivalent in its strengthening effect to the addition of double the area of cast iron, to say nothing of the part played by the reinforced concrete sheathing of which the bars in question form part.

The intrados slab of reinforced concrete constitutes a complete vaulting extending from side to side and from end to end

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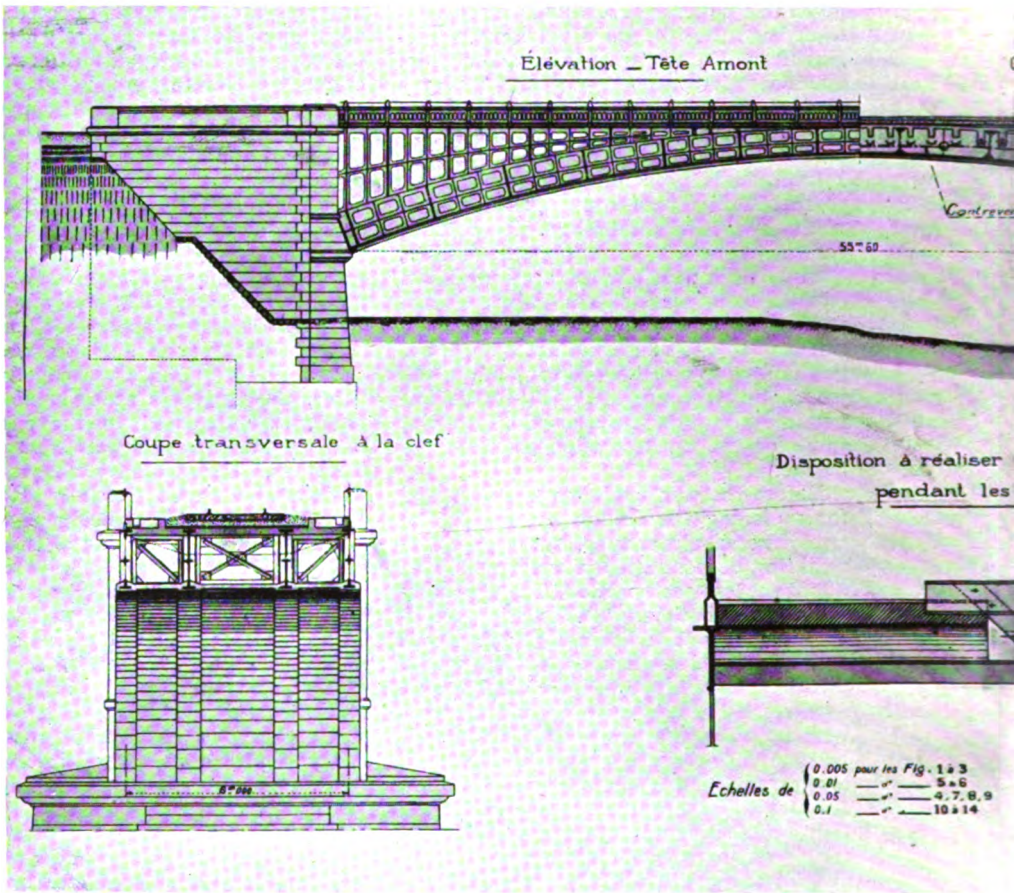

Top : Fig. 3.—Half Elevation and Half Longitudinal Section through Railway

Bottom Centre : Fig. 6.—Method of Support

Bottom Left : Fig. 5.—Cross Section at Crown of Arch, after Strengthening.

Strengthening a Cast-Iron Railwa

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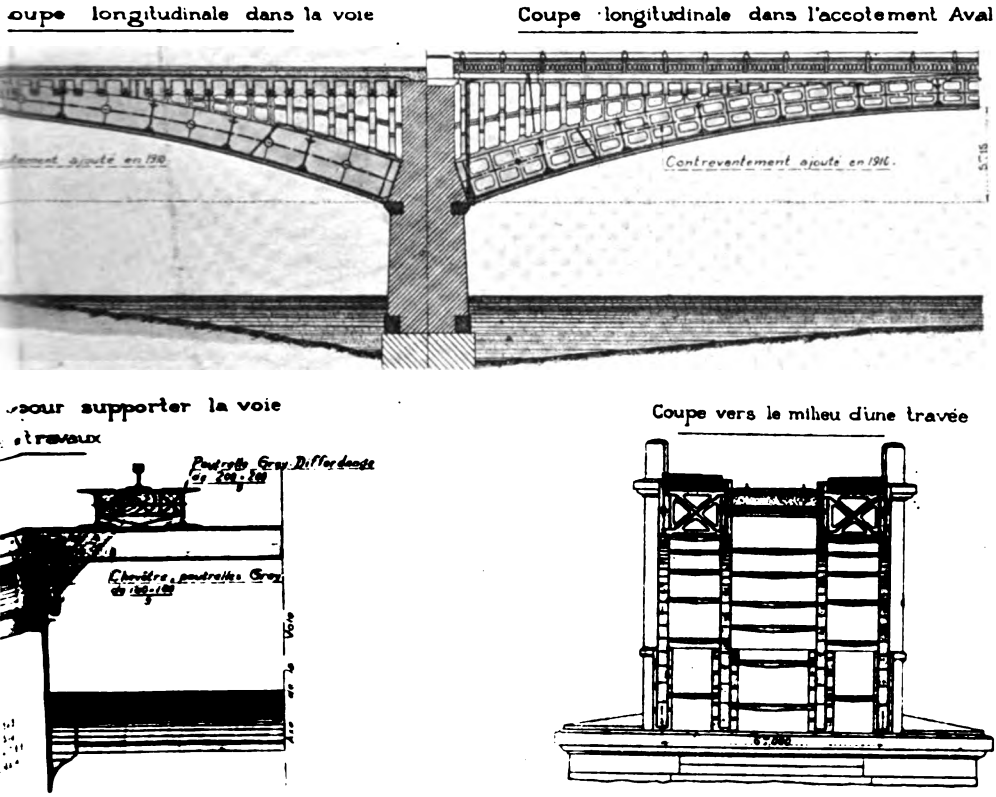

Railway track and Half Longitudinal Section through Footpath. All after Strengthening.
Supporting the Railway Track during Execution of Work.
Bottom Right: Fig. 4.—Cross Section near Crown of Arch, before Strengthening.

Railway Bridge with Reinforced Concrete.

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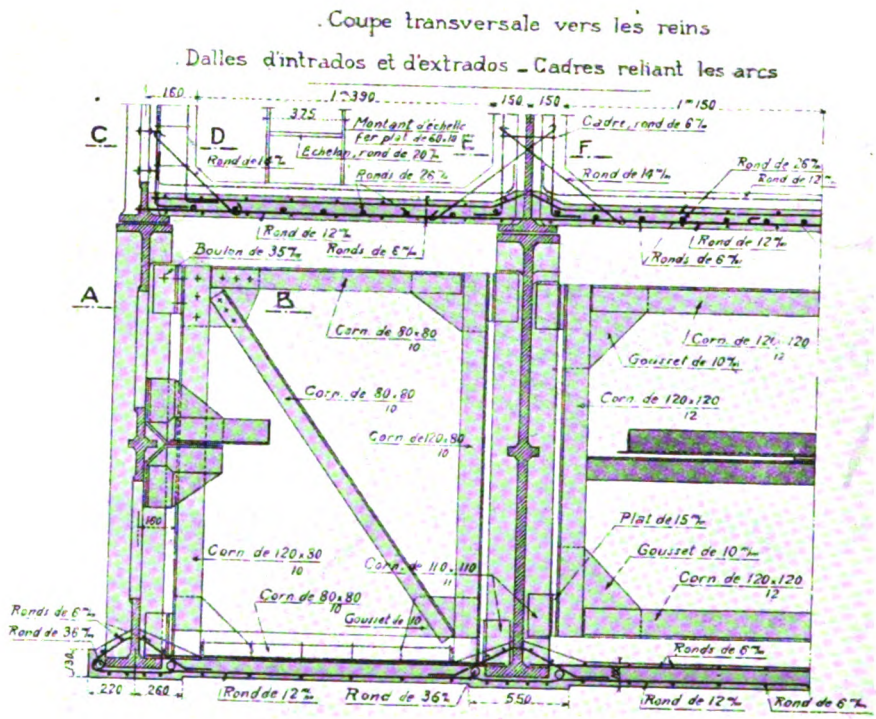

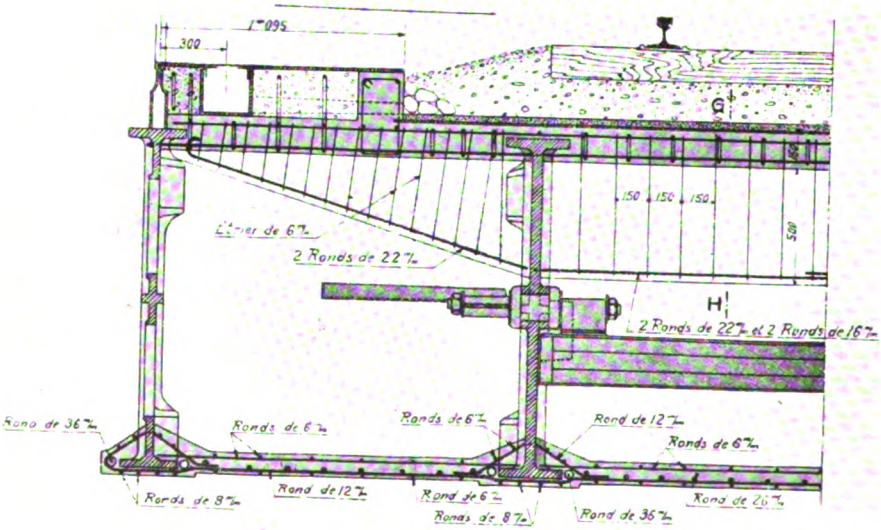

Top : Fig. 7.—Half Cross Section near Haunches, after Strengthening.
Bottom : Fig. 8.—Half Cross Section at Crown, after Strengthening.

Strengthening a Cast-Iron Railway Bridge with Reinforced Concrete.

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of each arch span, as shown by *Fig.* 2, its thickness being 10 cm. except where it is thickened for the strengthening of the arch rib lower flanges. The reinforcement is in two layers, near the upper and lower surfaces (see *Figs.* 7 and 8). The upper layer is formed of 6-mm. rods spaced at intervals of 20 cm. in two directions, parallel and perpendicular to the axis of the bridge. The lower layer also comprises two series of rods. Those laid transversely across the bridge are of 12 mm. diameter, spaced 20 cm. apart, and conforming with the spacing of the ties around the main bars for strengthening the lower flange of the arch ribs. The rods are bent down so as to pass under the bottom flanges and are secured to the 12-mm. ties of the main bars. The longitudinal rods of the lower layer are 6 mm. diameter, spaced 20 cm. apart. At the crown of each arch slab additional longitudinal reinforcement is provided in the form of 26-mm. bars at the same spacing. An important incidental advantage of the continuous intrados slab is that it constitutes a platform from which future inspections of every part of the bridge can be made without the need for scaffolding.

A reinforced concrete slab has been added near the springings of each span at extrados level of the arch ribs. Each of these slabs, 10 cm. thick, extends from side to side of the bridge and for a length of 6·50 m. from each springing; the reinforcement consists of 6-mm., 12-mm. and 26-mm. rods arranged in two layers and in two directions.

The reinforced concrete slab at deck level, 16 cm. in thickness from end to end (*Figs.* 5, 7 and 8), provides convenient and rigid support for the permanent way, braces the arch ribs and spandrel members, and being stiffened transversely by beams or ribs spaced 1·10 m. apart it transfers part of the superloads to the outer arch ribs, thereby relieving the two inner ribs from undue loading. The reinforcement of the slab and ribs consists of 12-mm. and 22-mm. bars; the latter pass through all four arch ribs, wedged in the intermediate ribs, and are attached to the outer ribs by bolts.

As it was not practicable to attach the ends of the deck slab to the masonry of the piers and abutments, owing to slight movements due to temperature changes, stability in the transverse direction was secured by encasing six of the transverse spandrel frames near each springing in reinforced concrete.

With the objects of facilitating the execution of work and permitting the construction of the new deck slab at a level suitable for ensuring its proper connection with the arches and framework, the rail level was raised by 60 cm. as soon as work was commenced and left at the higher level permanently, as will be seen by reference to *Fig.* 8 and by comparison of *Figs.* 4 and 5.

The construction of the deck slab and the alteration of the permanent way level was carried out without any interruption of traffic. While operations were in progress the permanent way was supported by temporary girders under the rails, and these in turn were carried by cross joists bearing upon the arch ribs and spandrel frames (*Fig.* 6). The raising of the permanent way rendered necessary a corresponding raising of the footpaths, and this led to an increase in the height of the parapets by the addition of round rails carried by cast-iron supports.

In order to promote satisfactory adhesion between the cast-iron work and the concrete the surfaces were thoroughly cleaned by sand-blasting. All the concrete used was composed of Portland cement, sand, and pebbles, and was mixed in the proportion of 1 : 1·6 : 3·2 by volume. The concrete was thoroughly rammed and kept covered for ten days with bags watered every morning and evening.

Three tests were conducted, as follows :—

(1) Before strengthening : Rolling load consisting of light locomotives.
(2) After strengthening : Rolling load consisting of the locomotives used in the first test.
(3) After strengthening : Rolling load consisting of "Pacific" type locomotives.

The results of tests (1) and (2) show that under the same loading the stresses in the cast-iron members after strengthening do not exceed one-third, and occasionally are not more than one-fourth or one-fifth, of the stresses occurring at the same points before strengthening. When the bridge was tested under the rolling load of three "Pacific" locomotives coupled together and run across twice at a speed of 28 miles an hour no appreciable vibration

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was felt. The results of the three tests proved conclusively that the anticipations of the engineers responsible for the strengthening scheme had been more than fully realised, and the structure is now able to carry the heaviest rolling stock and traffic of the Company. This result has been secured by the application of novel methods, resulting in a very important saving of cost, while not impairing the characteristic features of the original design. The total expenditure was 2,000,000 francs, whereas the reconstruction of the bridge with steel girder spans would have entailed an outlay of from 4,500,000 to 5,000,000 francs.

In view of the satisfactory structural and financial results attained by the methods described, the railway company felt no hesitation in proceeding with the repair and strengthening in a similar way of another bridge across the Rhone, near Lyons, a cast-iron structure of five arches.

A Substitute for Trass.

UNDER the name "Si-Stoff" a new material for use with cements has been put on the market in Germany. "Si-Stoff" is a fine, flour-like material, yellowish to ruby in colour, with a weight of 31·5 lbs. per cu. ft. loose and 47 lbs. per cu. ft. packed. Its chemical composition is :—

	Per cent.
Water	10·91
Silica (SiO_2)	64·54
Alumina (Al_2O_3)	15·85
Lime (CaO)	0·51
Iron oxide	2·90
Magnesia (MgO)	0·59
Sulphuric anhydride (SO_3)	3·87
Alkaline residue	0·83
	100·00

Its principal uses are to make mortars capable of resisting the action of water and acid. Some interesting results of experiments carried out at Dresden and by Professor Kayser at Darmstadt are given in *Tonindustrie Zeitung*. "Si-Stoff" converts ordinary white-lime into hydraulic cement and, as the table shows, increases its resistance to compression.

With Portland cement which contains free lime, whether owing to poor burning or not, "Si-Stoff" is claimed to give results closely resembling those obtained with trass. With Portland cement not containing free lime any increase of resistance obtained by the addition of trass or other material is purely due to the fineness of grinding of the added material. "Si-stoff" is ground very much finer than trass.

COMPRESSION TESTS AT 28 DAYS.

	Stored in water.	Stored in air.
	lbs. sq. in.	lbs. sq. in.
Slaked lime without "Si-Stoff"	59·8	179·0
25% (by wt.) lime + 75% "Si Stoff"	790·0	946·0
50% ,, + 50% ,,	1475·0	1362·0
Lime, burnt and ground, without "Si-Stoff"	(crumbled)	220·0
25% lime, burnt and ground, + 75% "Si-Stoff"	1720·0	1260·0

The tensile strength of Portland cement to which "Si-Stoff" has been added and which has been submitted to the action of sulphuric acid is shown in the following table :—

Mortar, 1 : 3	Tensile Strength (lbs. sq. in.).		
	1% H_2SO_4	2% H_2SO_4	3% H_2SO_4
Portland cement without "Si-Stoff"	243	146	71
,, ,, + 10% ,,	344	300	186
,, ,, + 20% ,,	443	368	205

Note.—Test specimens were kept one day in a moist closet, 13 days under water and 14 days in acid.

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DESIGN OF FORMWORK FOR REINFORCED CONCRETE CONSTRUCTION.

By A. E. WYNN, B.Sc., A.M.Am.Soc.C.E.

VI.—DETAIL CONSTRUCTION OF FOOTING FORMS.

In previous chapters we have considered the questions of loads and stresses and the method of calculating and choosing the correct timber sizes and spacing in order safely to carry the loads.

We now come to the practical details of assembling and erecting the forms for the various members that constitute a reinforced concrete structure. The methods that will be described are, of course, not the only ones, but they are the results of years of development and have become recognised as standards and are used by the majority of large concrete contracting firms.

The two main thoughts governing the methods of construction are (1) ease of stripping, and (2) unit construction.

The method of stripping, although it is the last operation, is the first consideration, since that is where time and money can be saved in both labour and material.

By unit construction is meant the building up of complete units, such as beam and column sides, wall and slab panels, etc., at the bench away from the points of erection, ready for erection as required. By this means most of the carpenter work is done under the best conditions with the aid of a power saw, leaving little to be done on the job where working conditions are not so good except the fitting together of the various units. Also, the building of the units can be started in advance of the date they will be required, so that there will be no delay in operations. Of course on a small job, where the forms will only be used once, it may often be more economical to build and erect the forms at the same time at the place where they are required.

Estimating Cost.—This subject would fill a book itself, and only sufficient information can be given to form a general guide and to illustrate a satisfactory method of estimating. Labour cost varies greatly, but for an average size job, using the methods of construction given here and with good organisation, the figures should not vary much from those given. Small jobs will cost more and large jobs less, as they can be better organised.

Each contractor should keep his own costs on performing different operations. This is easily done by having the timekeeper, or preferably the carpenter foreman, report at the end of each day the number of hours spent on all the operations, and at the end of each week or month the work done is measured up and the cost calculated.

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It is convenient to have a unit of measure for timber regardless of its size, and to apply an average cost per unit. This unit will be the cubic foot, and the amount of timber required to build any forms will be expressed in the number of cubic feet required per square foot of *contact with the concrete*, allowing for matching, waste, cleats, etc. Note it is the contact area that is measured and not that of the forms, since the first is definite and can be taken from the plans.

The amount of timber required per square foot does not vary much with the size of a member, so that the average value can be applied to all similar members regardless of their size. This amount will consist of about 1 part of sheathing to 1 to 1½ parts of timber, which determines the average cost per unit.

Labour cost is best calculated from the number of man-hours required to build the various kinds of forms and applying the current labour scale for carpenters and labourers. The number of man-hours must be divided between carpenters and labourers, and will vary in different parts of the country according to the rules of the trade unions. It will be assumed that the labourers will do all carrying and handling of timber, help in erection, and do most of the stripping. Timber piles will be assumed to be within 200 ft. of the place of assembly.

Wall Footing Forms (*Fig.* 8).—These are the simplest of all forms to build, being merely a heavy plank or light built-up panel set to line and staked in position.

Old timber, if available, is usually used, old 12 in. plank (or less) is very suitable for depths less than 12 in. One side should be set to line and held with stakes, about 6 ft. apart, the other side being set from this by 1 in. by 4 in. spreaders, cut to the width of the footing and knocked out as concreting reaches them. New planks should not be used for this purpose.

For deeper footings up to 2 ft. 6 in., which is about the maximum, the sides can be made of 1-in. timber built into panels about 8 ft. long, with battens 2 ft. on centres to hold the boards together. For heights up to 18 in. use 1 in. by 4 in. battens nailed on flat; for heights from 18 in. to 30 in. use 2 in. by 4 in. battens nailed on flat. The panels can then be held to line by 2 in. by 4 in. wales top and bottom, which can be braced back to stakes or the sides of the trench about every 6 ft.

The side panels need not be the exact depth of the footing, but should be as much greater as is necessary without sawing the boards. To space and line one side from the other 1 in. by 4 in. spreaders are used. Sometimes the trench is backfilled against the forms, which saves the bracing although it increases the cost of stripping. The forms can be stripped after a few hours and moved forward.

Column Footing Forms.—There are three standard types of column

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or pier footings: the square or rectangular footing, the stepped footing, and the sloping side footing, the two latter being variations of the former in order to save concrete. They are all built in a similar manner of 1 in. tongued and grooved sheathing cleated together, the four sides being built as units, assembled at the place of erection, and fastened together. Cleats or battens should be 24 in. apart, and the size depends on the depth of the footing, using 1 in. by 4 in. up to 18 in. deep; 2 in. by 4 in. for depths from 18 in. to 30 in.; and 2 in. by 4 in. on edge for depths from 30 in. to 48 in.

Square or Rectangular Footings (*Fig.* 9),—There are two ends

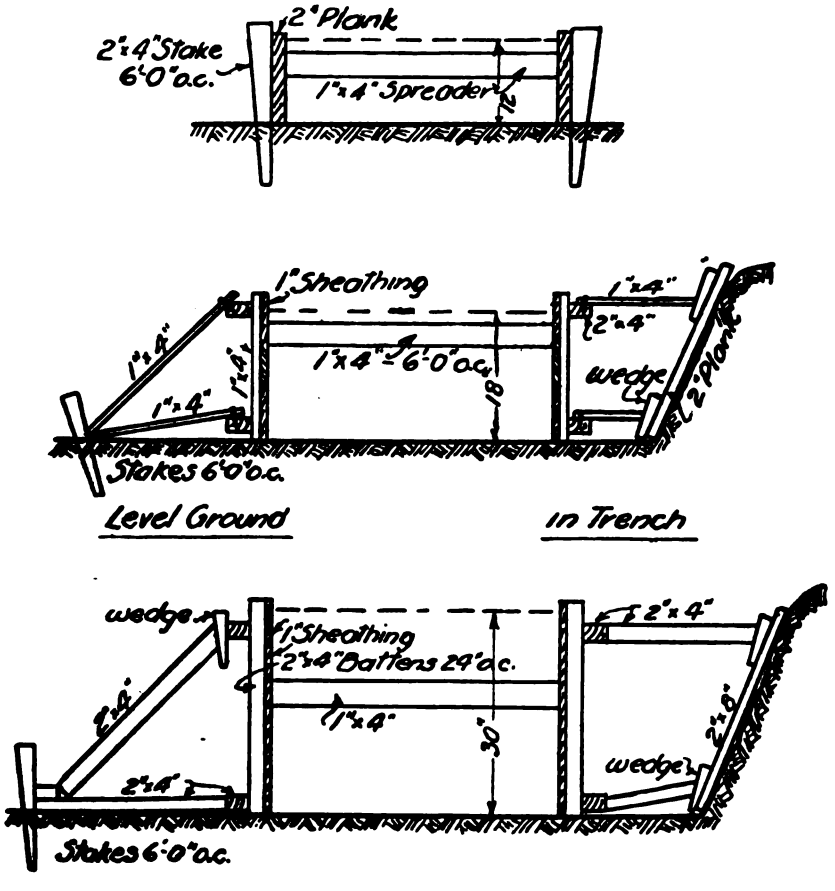

Fig. 8. Wall Footings

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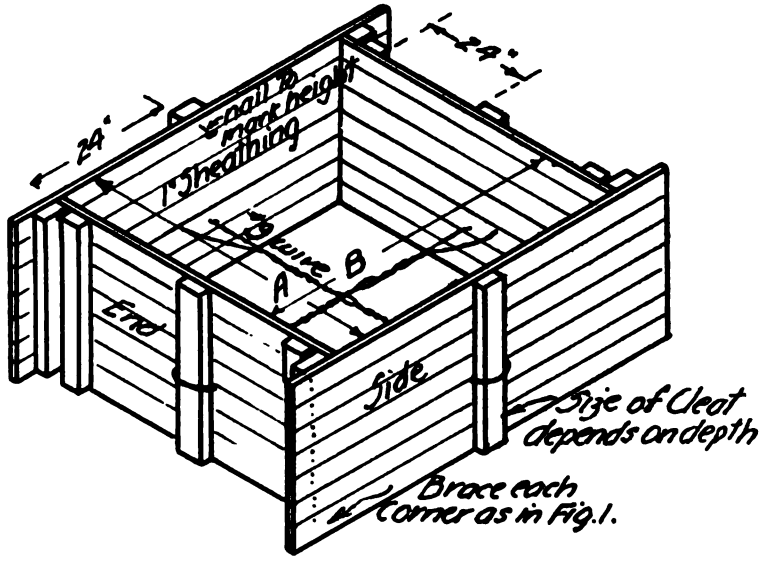

Fig. 9. Square Footing

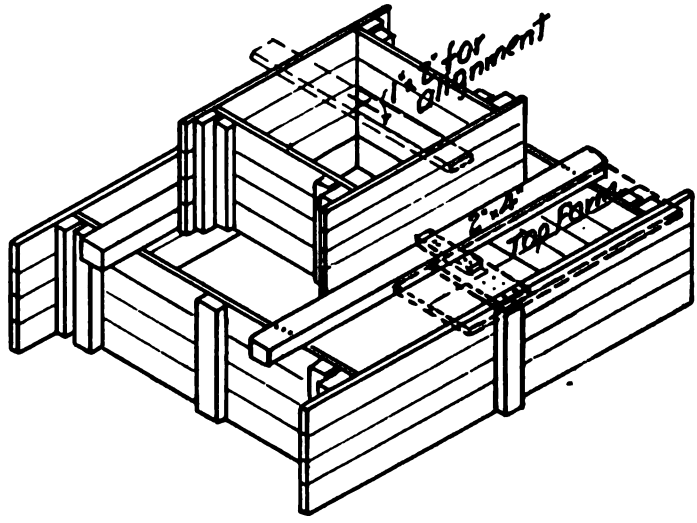

Fig. 10. Stepped Footing

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and two sides, and the ends are the short sides of rectangular footings. The ends are built to the exact dimension (A) of the footing, with a cleat each end on the outside placed a distance from the ends of the sheathing equal to the depth of the cleat used with a slight allowance for clearance; that is, if the cleats are 2 in. by 4 in. dressed the cleats on the ends will be 2 in. from the ends of the sheathing. The two sides are built about 12 in. longer than the dimension of the footing, although all the boards need not necessarily be sawn to the same length, and are cleated each end on the inside, the distance between cleats being the dimension (B) + twice the thickness of the sheathing, using the dressed thickness.

One or two intermediate outside cleats will be necessary on sides and ends, so that the distance between cleats is not over 24 in. The size of the cleats will depend on the depth of the footing, as above.

The side and end panels should be made up on the bench, with holes bored for the wires if they are used, all ready to be carried to the place of erection when required.

The four panels are assembled by butting the end panels against the end cleats on the sides. If they are to be used only once or twice the panels can be nailed together through the cleats; if this method is followed the end cleats on the end panels are sometimes omitted, except one cleat at the corner which is to be opened for stripping. This corner is nailed together from the outside with long spikes through both cleats, and to strip only this corner is opened sufficiently far to enable the form to be removed as a whole.

This cannot be done successfully with very large forms or when the forms are to be used several times over, in which cases the panels should not be nailed together. The corners should be braced back to stakes or to the side of the excavation as in *Fig.* 8, bracing only being required on the sides. The intermediate bracing can be done in the same way, or opposite cleats can be wired together as in *Fig.* 9. The wires should be placed about a third of the way up and should consist of a double strand of No. 9 or 10 wire, and if the footing is over 3 ft. deep two double strands should be used. The corners can also be held by wiring together the end cleats on the sides, and no exterior bracing need be used. When wires are used the reinforcing steel must be placed before the wires.

No labour should be spent in cutting the panels to the exact depth; full width boards should be used, and a nail driven in the side to show the depth of the footing.

Stepped Footings (*Fig.* 10).—To save concrete a deep footing is often built in two or three steps of decreasing size. Each step will be similar to the form for a square or rectangular footing described above, the only point to note being the method of placing and supporting the successive forms.

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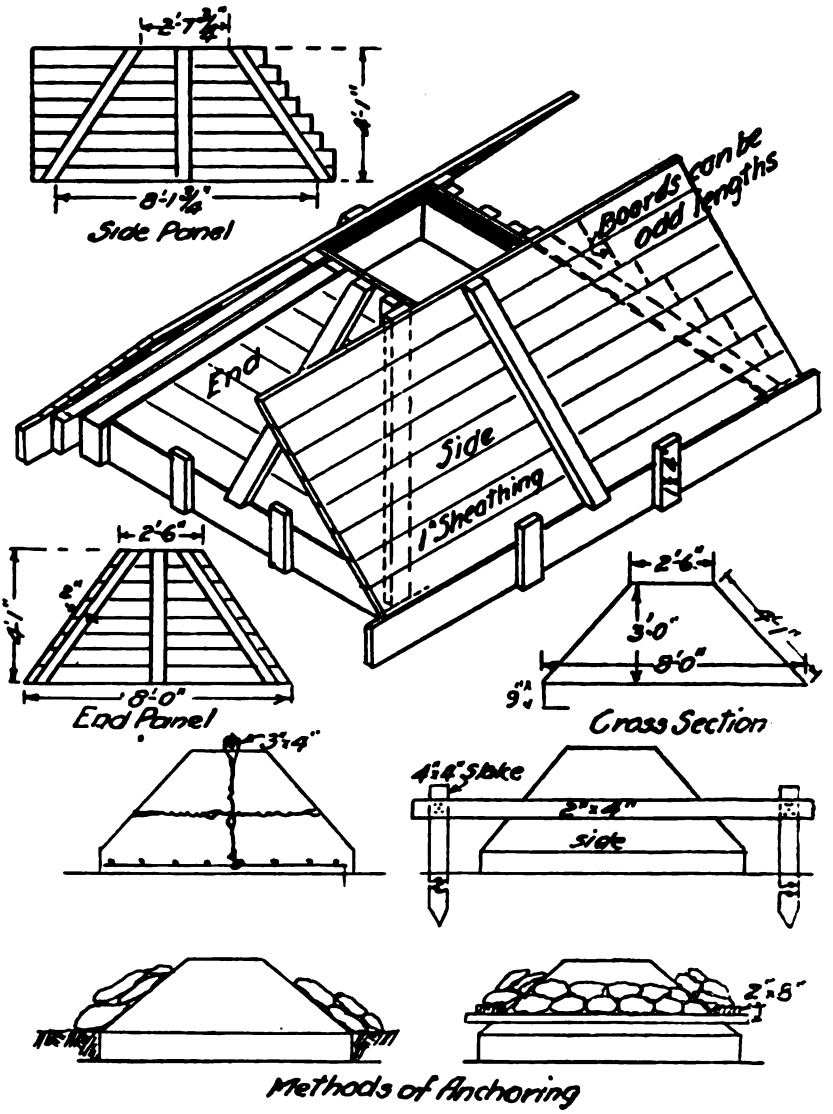

Fig. 11. Sloping Side Footing

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The largest and lowest form is placed first, and usually filled and the concrete allowed to set a little before placing the next form. The upper form can then be held by placing a 2 in. by 4 in. on each side, nailed on to the top of the sides of the lower form.

If it is desired to pour all the steps together, the space between the different forms must be sheathed over with a top form to prevent the concrete from overflowing. This top form is shown in dotted lines in *Fig.* 10; it is built out of two boards cleated together and held down by cleats to the 2 in. by 4 in. As there will be considerable upward pressure on the top form it should be well weighted down.

Sloping Side Footings (*Fig.* 11).—Footings with four sloping sides are built in the same manner as when the sides are straight. The two ends are cut to the exact dimensions of the footing, first laying out boards of approximately equal length, then marking out the slope lines, putting on the cleats, and sawing through on these lines. The two sides should not be cut to the slope, as this is unnecessary, but the boards can be left of random lengths. Two inside end cleats are nailed on parallel to the slope, allowing a clearance each end from the exact dimension of the side equal to the thickness of the sheathing.

The sloping box is always set on a shallow straight box of the same bottom dimensions. The straight box is set first on the ground, securely braced, often by backfilling around it, by wires or by braces to stakes. The reinforcing steel is placed and the sloping form set on top of the box. Its alignment will be fixed by that of the straight form, the cleats on which should extend a little above the sheathing to hold the sloping form.

It is very important that the form be prevented from lifting due to the pressure of concrete on the sloping sides; considerable trouble will ensue if this is neglected. To prevent this uplift a wire or wires can be embedded in the centre of the lower portion, fastened to a spike in the piles if piles are used, or to the reinforcing steel if heavy enough to resist the uplift, and twisting the loose ends around a 3 in. by 4 in. across the top of the form. A few bags of sand or cement or heavy stones placed on the sides will answer the same purpose, or if the ground is firm and dry a 2 in. by 4 in. can be nailed halfway up the form on each side, nailing the ends to stakes in the ground—stakes will not hold in soft ground.

It will be necessary to calculate the height to build the sides and ends, as only the top and bottom dimensions and the height of the footing will be given. It can easily be found by laying out two opposite cross sections of the footing and measuring the length of the sloping sides; these lengths will be the heights to build the panels. The sides should not be built the exact heights, but whatever the boards

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give without sawing, and the height of the concrete marked by a nail.

Alignment of Footing Forms.—A mason's line or fine wire should be stretched taut along the centre line of the footings, attached to stakes preferably outside the building. The spacing of the footings can then be measured off, tying on a piece of line at each centre, or preferably, if there are a large number of footings, by stretching a similar line through at right angles. Across the top of the form nail a 1 in. by 2 in., mark off the centre line and the centre on it, and adjust the form until these marks coincide with the lines above.

Cost of Footings.—The amount of timber required is the first consideration. There are usually several footings alike, and in an ordinary building only a few different sizes, so that it is only necessary to build forms for a portion of the footings and reuse them, the number depending on the speed required and the number of different sizes. Footings can be stripped the day after they are poured.

When there are several similar-shape footings differing only in size, if the forms are made up for the largest they can easily be reduced for the smaller sizes by sawing a strip off each end panel and putting new stop cleats on the side panels. Most forms can be used three or four times over without any loss in time.

Allow $\frac{1}{6}$th cubic foot of timber per square foot of contact area of all the footings and divide this amount by three or four according to the number of times they are to be used over, or if the sizes differ considerably base this amount on so many of the largest sizes.

To make up panels, erect, and strip 100 sq. ft. of forms will require: For making up, 3 hours carpenter time and $1\frac{1}{2}$ hours labourer time; for carrying to place and erection, 2 hours carpenter time and 2 hours labourer time; for stripping, 3 hours for either carpenter or labourer, usually labourers.

Cost per 100 sq. ft. if used once = 5 hrs. carpenter at.....
$6\frac{1}{2}$ hrs. labourer at.....

Timber (used once) $= \frac{100}{6}$, say, 17 cu. ft. at.....

The labour cost is based on the total area in contact.

If the forms are used more than once the cost of making up the panels is divided by the number of times used and added to the cost of stripping and erecting.

Cost per 100 sq. ft. if used 4 times = $2\frac{3}{4}$ hrs. carpenter at.....
$5\frac{3}{8}$ hrs. labourer at.....

Timber used 4 times $= \frac{100}{24}$, say, $4\frac{1}{4}$ cu. ft. at....

(To be continued.)

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The Foreman's Guide to Concrete.—VI.

A MANUAL FOR ALL ENGAGED IN CARRYING OUT CONCRETE AND REINFORCED CONCRETE STRUCTURES.

By Albert Lakeman, M.S.A., M.I.Struct.E.

REINFORCED CONCRETE BEAMS.

APART from the types of beams previously given, the design is affected by the condition at the ends which may be supported, restrained, or fixed according to circumstances, and this condition is of considerable importance because not only is the capacity of the beam influenced, but the stresses in the beam as regards character are also affected.

Beams and slabs are divided into different types, as follows :—

(1) Supported only at both ends ;
(2) Fixed at both ends ; and
(3) Supported at one end and fixed at the other.

These types are distinct from the cantilever or bracket, which is obviously fixed at one end only with no support or assistance at the other end. Beams and slabs which are continuous over several spans will be equal to those having fixed ends, because the ends of each span will be held in position by the adjoining bay and they are not therefore free to move as in the case of an end which is simply supported.

SUPPORTED ENDS.

Dealing with the most simple type first, the condition when the ends are supported only is shown in *Fig.* 36, where a beam is indicated carrying a concentrated central load. The tendency for the load to cause bending or deflection is shown in an exaggerated manner, and it will be seen that the ends of the beam are free to move so that the whole beam becomes a part of a circle with the whole of the top part in compression and the whole of the lower part in tension. This condition will apply to beams and slabs only where the ends are supported ; the condition of loading, whether concentrated or distributed, will not make any difference in the *nature* of the stress but only in the value.

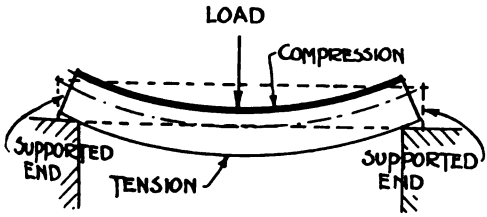

FIG. 36

FIXED ENDS.

If, however, the ends of the beam are held firmly in position, then the beam is unable to bend in one simple curve under the action of the load, and the tendency is to take the shape shown in an exaggerated manner in *Fig.* 37. In this diagram both ends are fixed and a portion of the beam at each end acts as a cantilever with the top surface tending to stretch and the bottom surface tending to contract, while the central portion acts as a beam between the two cantilever portions. The shape of the beam is therefore a combination of three curves, and as the two end curves radiate from a centre below the beam while the middle portion radiates from a centre above the beam, it follows that the tendency to compress the top surface, which occurs with a supported beam, does not apply to the end portions of a fixed beam. The fixing of the ends therefore results in tension and compression on the top surface (viz. tension at the ends and compression in the centre portion), and the reverse stresses on the bottom surface (viz. compression at the ends and tension in the centre).

In the diagram the portion under compression is shown by a thick line, and the portion under tension by a thin line.

The stress therefore changes in character at some point in the length of the beam, and this point is called " the point of contraflexure." The position of this point will depend on the type of load which will cause a slight variation, but for all practical purposes it can be taken as one-fourth of the clear span from each end. This is shown in the diagram as $\frac{1}{4}l$ when l equals the span, and the point of contraflexure is indicated by B. It will be realised that this change of stress is of the greatest importance, because it is

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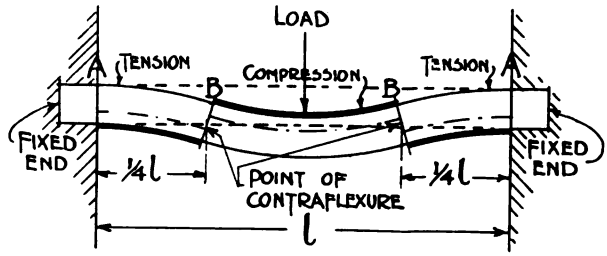

FIG. 37

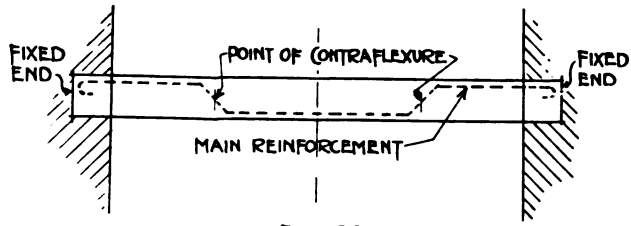

FIG. 38

necessary for the reinforcement to be placed in the tension area of the beam, and therefore in the beam shown in *Fig.* 37 the rods must be placed on the upper surface between the points *A* and *B*, while they are required on the lower surface between the two points marked *B*. In order to arrange this, the rods are cranked at the points of contraflexure as shown in *Fig.* 38.

The fixing of the ends reduces the maximum stresses in the beam because it has the effect of theoretically reducing the span, and therefore this method is adopted in practice whenever possible. The effect of fixing the ends, as compared with the condition of supported ends only, can be easily tested by experimenting with a small beam made with rubber, or even with a piece of wood, say, about 2 in. square, laid across two supports some distance apart. By leaving the ends free in the first case and putting a weight on the piece of wood sufficient to cause bending, the shape can be noted, and if the weight is then removed and the ends firmly secured by nailing down and the weight again applied it will be found that the piece of wood is much more stiff, and when bending does occur the shape is quite different from that obtained when the ends were free.

ONE END FIXED ONLY.

It is not always possible to fix both ends of a beam in practice, but it may be possible to fix one end while the other is supported only, and in this case the condition will be as shown in *Fig.* 39. The action of the load in this instance tends

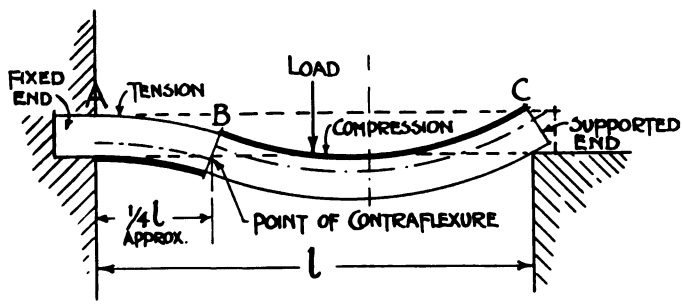

FIG. 39

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to bend the beam to the shape shown in this diagram, again exaggerated, the portion *A–B* being under tension on the top surface and the portion *B–C* being under compression on the upper surface, owing to the part near the fixed end acting as a cantilever and the remainder acting as a supported beam. The shape consists of two curves instead of three as is the case with both ends fixed, and the point of contraflexure or position at which the curve changes is situated at 0·273 *l*, or, for practical purposes, say, one-quarter of the span from the fixed end. The reinforcement in this type of beam will require to be placed in the upper surface between the points *A* and *B*, and in the lower surface between the points *B* and *C*, with a crank at the point of contraflexure.

CONTINUOUS BEAMS.

In reinforced concrete floors it is a frequent occurrence to find beams which are continuous over several spans, the intermediate supports consisting of reinforced concrete columns, and in this case the length of beam between any two columns can be considered as restrained or fixed at the ends and calculations made accordingly. In the case of the end span this will generally be taken as supported at one end when it is resting on a wall and the other end will be considered as fixed. The tendency of a beam which is continuous over several spans is shown in *Fig.* 40, where the bending is indicated ; the thick lines give the portions under compression and the thin lines the parts under tension. It will be seen that the stress is alternately compression and tension in both top and bottom surfaces owing to the cantilever action on each side of the column support.

The importance of this aspect of the design of reinforced concrete is considerable, because the calculations are based on certain conditions, and allowance is made for the increased strength due to the fixed ends which must be given in the executed work. The reinforcement must be cranked exactly as shown on the drawings, the continuity of the rods must be given by strict adherence to the details supplied by the designer, and any special rods provided in the upper surface to take the tension over the support must be accurately placed. Again it will be necessary to take particular care to ensure the ends of the beam being properly fixed when provision is made for this condition, otherwise the bending tendency will be greater than that considered in the calculations.

In some schemes, when concrete beams are used in conjunction with brickwork, a beam is designed as continuous running across one or more brick piers, and when this is the case it is essential that the brick piers be built right up to the finished level before the formwork is fixed and the beam executed, otherwise the removal of the formwork and props preparatory to the building up of the piers will result in a tendency for the beam to deflect or sag in one large curve and cause tension stress in the whole of the under surface, whereas the design has been made to provide for compression at the points where the piers occur.

In addition, the span of the beam taken in the calculations will be the distance between two piers when they are erected complete, and the omission of support at the pier positions while the concrete is green means that the beam is called upon to carry itself and any load across a much larger span than provided for in the design.

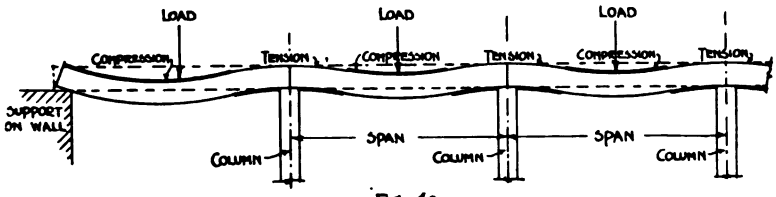

FIG. 40

CONTINUOUS TEE-BEAMS.

It will be realised that in the case of tee-beams it is not advantageous to fix the ends, because it is necessary to keep the whole length of the upper surface of the beam in compression to allow the slab forming the top flange to take the stress. If the ends of a tee-beam are fixed or restrained and compression has to be pro-

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vided for in the lower surface of the beam at the ends it is necessary to increase either the concrete or the steel, and in some cases both must be added to. The increase of the steel will not be economical owing to the low limit of stress that can be allowed on the steel in compression, and it is better to increase the concrete if this is practicable.

This increase is sometimes provided for by making the beam deeper by a splayed portion near the ends, as shown in *Fig.* 41. The increase in the depth increases the resistance of the beam and compensates for the loss of the assistance of the flange, and it is the best way of providing the necessary concrete unless the splayed portion required is excessively large and

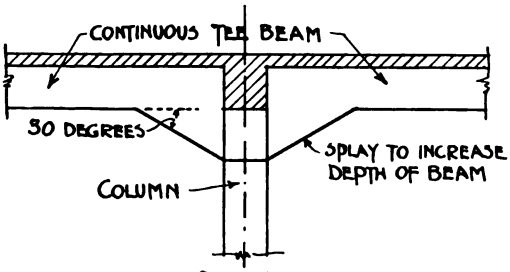

FIG. 41

interferes with the headroom below the beam. The regulations of the L.C.C. provide that where the end of a beam is splayed for the purpose of increasing the resistance moment, such splayed work shall not be calculated at a greater angle than 30 degrees from the horizontal. This restricting angle is shown in *Fig.* 41, and if the necessary resistance cannot be obtained by working to this increase in depth then the balance must be made up by the addition of steel in the compression area if the ends of the tee-beam are fixed. It should be realised that these splayed portions are provided for the purpose of giving the strength required to support the calculated load; they are not added for the sake of effect, and must therefore be executed with the same care as the remainder of the work. When placing the concrete for the column the work should be stopped at the level of the bottom of the splay in order to allow the full depth of the beam to be concreted in one operation.

LEVER ARM.

Before proceeding to deal with the variation of the bending moment and stress in a beam it is necessary to explain the effect of leverage and what is known as the "lever arm," because the variation is due to the variation in the length of the lever arm, and without some knowledge of this factor the reader will not be able to understand fully how the bending moment is calculated by the designer and why it has a different value at different points in the beam. Levers are in common use on every building scheme; the labourer will use a lever to overcome the resistance of a weight which is otherwise beyond his power to move, without having the least technical knowledge or idea of the theory which he is illustrating.

There are three kinds of levers, known as the "three orders of levers," and each of these has its own particular value as regards the power necessary to overcome a specified weight or resistance, and the whole of the calculations for a reinforced concrete building are based on one or other of these levers, although this may not be apparent even to the designer.

The most simple illustration of a lever is that given by a labourer who takes a crow-bar, and, placing a piece of wood or a brick on the ground, puts his crow-bar across this to lever up or move a heavy weight, as illustrated in *Fig.* 42. This is a lever of the "first order," because the lever turns on the edge of the brick, which is known as the "fulcrum" (the fulcrum meaning the point which acts as the hinge for the lever), and this fulcrum is situated between the weight and the pressure.

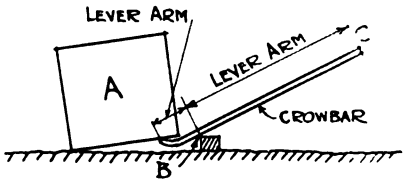

FIG 42

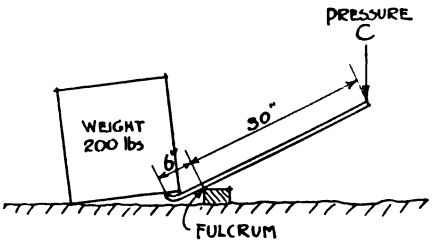

FIG 43

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In the second order the weight is situated between the fulcrum and the pressure, and in the third order the pressure is placed between the fulcrum and the weight. It does not matter what is used for the fulcrum provided it is strong enough to resist crushing under the load. The lever can also be any material capable of offering the necessary resistance to bending and breaking under the load. The object of the lever is to get a mechanical advantage which will increase the value of the power and the gain in power will depend on the comparative lengths of the two lever arms; that is to say, the comparative lengths of the lever on each side of the fulcrum which divides the lever into two parts. To give a definite example we can consider a block of stone weighing 200 lbs., and a crow-bar 36 ins. long with a brick for a fulcrum placed 6 ins. from the bottom end. The condition will then be as shown in *Fig.* 43, which will be recognised as similar to *Fig.* 42 but with figures added. The only value that is not given in the diagram is the pressure required at *C* to lift the weight, and this can be calculated in a simple manner, because the weight multiplied by its distance from the fulcrum will equal the pressure multiplied by its distance also from the fulcrum, therefore, 200 lbs. × 6 ins. = pressure × 30 ins., or pressure $= \frac{200 \times 6}{30} = 40$ lbs. This means that if the workman will apply a pressure of 40 lbs. at the end of the lever he will balance the weight of 200 lbs. and any fraction of pressure over 40 lbs. will lift the weight. The advantage is gained because the lever arm of the pressure is 5 times as long as the lever arm of the weight.

It will be seen that the nearer the brick or fulcrum is placed to the weight and the longer the crow-bar, the less the pressure that has to be applied, and if this principle of leverage is fully understood, it will be possible to take advantage of this gain in power to the maximum.

The second order of lever is one where the weight is between the power and the fulcrum. An illustration of this is given in *Fig.* 44, which shows a crow-bar being used as a lever to move a rolled-steel beam by pushing it under the beam so that the lower end rests on the ground, and the power is applied by pulling up the crow-bar at *C*. In this example the weight (the R.S.J.) is shown at *A*, and the ground acts as the fulcrum as indicated at *B*, while the power is applied at *C*. The

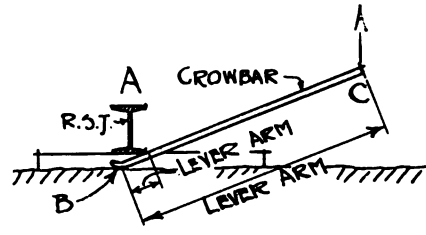

FIG. 44.

weight is therefore between the fulcrum and the power. The lever arm of the weight is the distance from the weight to the fulcrum, while the lever arm of the power is the distance from the power to the fulcrum, which in this type is equal to the whole length of the lever. The same principle of calculation applies as before, and if the weight of the R.S.J. is 200 lbs. and the distance from the weight to the fulcrum is 6 ins., as before, and the crow-bar is 36 ins. long, then the power required equals $\frac{200 \text{ lbs.} \times 6}{36} = 33\frac{1}{3}$ lbs.

It will be seen that the power necessary to lift the same weight as that given in *Fig.* 43 is less although the total length of the crow-bar is the same and the distance from the weight to the fulcrum is the same, and this shows that a greater advantage is gained by the use of a lever of the second order than with that of the first order. This is an important point as showing how the application of theory to the practical work can be of assistance to the man in charge.

With the third order of lever there is always a mechanical disadvantage, because the power required will be greater than the weight to be moved, and this lever is therefore used for the purpose of convenience and not for an increase of power value. The power is placed between the weight and the fulcrum, and one of the best examples of this is provided by the illustration in *Fig.* 45, which shows the treadle of a sewing machine where the weight or resistance is at *A*, the fulcrum is the pin at *B* on which the treadle works, and the power is applied by the foot at *C*. In this case the lever

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arm of the weight to be moved is equal to the whole length of the lever, while the lever arm of the power is much less and

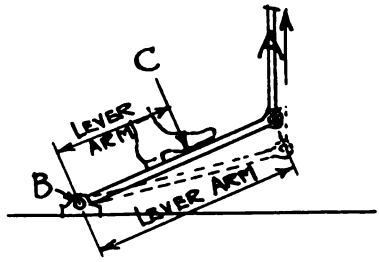

FIG. 45.

will generally be only about one-half the full length of the treadle. If the length of the treadle is, say, 24 ins., and the weight to be moved equals 20 lbs., while the pressure of the foot is applied at the centre of the treadle, which is only 12 ins. from the fulcrum, then the pressure $= \frac{20 \text{ lbs.} \times 24}{12} = 40$ lbs. The ratio of the lever arms in this case is 12 ins. to 24 ins., which equals one-half, and the pressure required is therefore equal to twice the weight. This order is sometimes called the "losing order" because there is a loss of power, but it is a condition which actually occurs in many parts of a structure. The principle of the third order is very important as it forms the basis of the calculations for reactions and other factors that have to be determined by the designer.

All concerned with the execution of reinforced concrete structures should carefully study these notes on levers and work out a few simple examples, and also make a few easy tests on the job by using a plain lever and different weights to prove the accuracy of the calculations and results.

(To be continued.)

The Use of Concrete in Gas Cleaning Plant.

A NEW application of concrete is proposed in connection with the electrostatic precipitation process of cleaning smelter and furnace gases, which has been extensively used in the United States of America for the recovery of potash, which is a valuable fertilizer, from cement kiln gases.

In this process a high-tension electrical discharge is passed through the gases, depositing the dust or other suspended matter in the fumes on the collecting electrodes, which were formerly constructed of large sheets of mesh metal, or of metal pipes. It has been found that a more evenly distributed discharge is obtained by substituting concrete collecting electrodes for the metal ones previously used, because their electrical resistance is nearer the resistance of the electrostatic field in the treater. It is proposed to vary the resistance of the concrete by suitably proportioning added substances of higher conductivity; for example, metallic oxides, such as magnetic iron oxide, or carbon in a finely divided state.

The employment of concrete in this particular instance is found to have further advantages, in that the collected impurities do not exhibit their usual tendency to adhere to the electrodes, but fall off automatically. In the past it has been necessary to make mechanical provision, such as rapping or scraping gear, for their removal. The electric discharge being more even and a higher voltage being maintained, since the electrodes are always clean, has the result of increasing the efficiency and capacity of the gas-cleaning plant.

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Concrete Roads at Gosport.

Costs and Methods of Construction.

At a recent meeting of the Southern District of the Institution of Municipal and County Engineers, Mr. E. J. Goodacre, Borough Surveyor, gave some useful information on the all-concrete roads which have just been completed in Gosport.

The majority of the roads in Gosport, including all the secondary roads, were, he said, constructed with local gravel. The district being flat, it was difficult to provide adequate surface drainage, and the rainwater remained on the crown of the road, forming potholes. These broke up the road very rapidly; in fact, the "life" of a gravel road in this district was not much longer than 12 months; it cost approximately 2*s*. 6*d*. per super yard, and then there was the cost of street watering in an attempt to keep down the dust. In many roads the service mains were so very shallow—sometimes not more than 18 in. deep—that it was quite impossible to use a steam roller without damaging them. In view of this, together with the fact that an excellent cheap aggregate was available, concrete was adopted for the construction of the new roads.

The aggregate for the lower course consisted of Langstone Harbour gravel and sand, which was delivered alongside the Corporation wharf for 4*s*. 6*d*. per ton. The top 1½ in. of concrete was ½ in. to ⅛ in. graded Alderney chippings, for which 17*s*. 6*d*. per ton was paid, f.o.b. Gosport. Cement cost 56*s*. per ton (subsequently reduced a little), f.o.b. Gosport. As this was a relief scheme, the mixing was done by hand under very stringent supervision.

The total thickness of the concrete was 6 in. The lower 4½ in. was in the proportion of 1 cement, 5 Langstone Harbour ballast and sand; the 1½ in. top layer: 1 cement, 1 sea sand, 3½ Alderney granite chippings. The whole was reinforced with B.R.C. fabric placed 1½ in. from the under surface of the concrete. Gauge boxes were used and water added by means of watering-cans with roses. Special precautions were taken not to get the mixture too wet; the consistency aimed at was a plastic one, permitting compacting and moulding with surface finishing, but without run off or separation of ingredients.

The method of laying was as follows:—The existing surface was excavated to the proper depth and carefully formed, the channel stones being taken up and removed. Screeds were then fixed inside existing kerbs, the gutter soles being concreted later after removal of screeds. The two layers were brought along together, and the bottom layer was never allowed to be more than 3 ft. ahead of the wearing surface; both were finished together on the same line at the end of the day's work. The whole of the surface was tamped by hand and finished with a crossfall of 1 in 40. It was found that the hand tamping gave a very desirable texture to the finished surface. No pretentious method of curing was carried out, but the surface was kept as wet as possible for several days, and covered with a thin coat of wet sand. No special measure against damage from expansion was used beyond that given by finishing the day's work with a vertical joint. Subsequent results had justified this method, as in the whole job—nearly 20,000 super yards—only one crack developed during 1923, when the surface temperature of the concrete road was 104 degrees Fahr., and this was laid during a temperature of 45 degrees Fahr. However, this part of the work was the first section to be done, and on examination the concrete proved weak in this particular spot. The reinforcement was quite unaffected.

The actual cost had been 9*s*. 6*d*. per yard super, as follows:—Labour, 1*s*. 11·2*d*.; granite chippings, 9·1*d*.; ballast, 1*s*. 0·1*d*.; sand, 4·0*d*.; cement, 2*s*. 7·3*d*.; reinforcement, 1*s*. 9·0*d*.; timber, water and incidentals, 0·2*d*.; making a total of 8*s*. 4·9*d*. The average cost of removal of surface was 12·4*d*., making up the total cost of 9*s*. 6*d*. per yard super.

It was difficult to estimate the "life" of these concrete roads, but anything over a four years' "life"—by all appearances to-day the "life" would be nearer 20 years—would be more economical than the previous gravel roads.

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SIZE AND CONFORMATION OF SANDS.

At a recent meeting of the Institution of Structural Engineers, Mr. M. Jackson, A.M.Inst.C.E., read a paper in which he outlined an investigation he had undertaken into the size and conformation of sands with a view to ascertaining their suitability for concrete.

In his opening remarks he said the method of specifying the proportions of concrete material by so many loose volumes of gravel to a definite number of volumes of cement had much to commend it, for it was easy of application. To those using concrete, however, it had been apparent for a long time that to get the best results those proportions must be determined by having regard to the nature of the aggregate. Many examples could be recalled in which a "lean" concrete had developed much greater strength than a much "fatter" one. A recent method of investigating the matter, especially by American engineers, was to determine how much mortar composed of 3 parts of sand to 1 of cement would fill the voids in the aggregate of broken stone or gravel and to specify the quantities accordingly. These methods were more or less empirical, and the lecturer's investigation had been undertaken to ascertain if possible some definite standard of aggregate material; a more or less ideal substance towards which one might approximate.

The investigation had been of a twofold nature. First, a theoretical examination of how particles of spherical form might be assembled—the angle of repose, the voids between the particles, and the amount of cement, or any other granular material of smaller size than the original material, required to cover the particles and to fill the voids. Second, an experimental examination of (*a*) a number of sands, to ascertain the size of the individual particles; (*b*) the shape of the particles, (*c*) the degree of approximation to the conditions of spherical material, (*d*) the determination of the angle of repose for particular size sands, (*e*) the voids for particular size sands.

In dealing with the theoretical examination of the subject, he said a heap of spheres could be arranged with each sphere touching adjacent spheres in three different ways. Drawing No. 1, Case 1, showed such a pyramidal heap with square formation base; one layer resting on the layer below by contact at the highest and lowest points. Drawing No. 1, Case 2, showed a heap of hexagonal formation in plan; each sphere fitting in the "V" between adjacent spheres. The layers of spheres rested on one another by contact at the top and bottom points, as in Drawing No. 1, Case 1. Drawing No. 1, Case 3, showed a heap of hexagonal formation in plan, as in Case 2. The layers of spheres rested on one another, each sphere filling the "V" space between three spheres immediately underneath it. Drawing No. 1 also showed the plans of the several layers with the positions of the spheres of the next lower supporting layer.

It would be seen that quite irrespective of the size of sphere the slope of the pyramid was constant for any particular formation. These slopes, which were analogous to, if not identical with, the angle of repose, had the following values:

	Minimum.	Maximum.
Case 1 . . .	35° 16′	45° 0′
Case 2 . . .	45° 0′	49° 7′
Case 3 . . .	58° 25′	61° 58′

It was a noticeable feature that Case 3 was the only case in which the spheres would not roll down into a lower position. In the other two cases each sphere was in unstable equilibrium, and yet the angle of repose in Case 3 was greater than in the other cases. It was noticeable, too, that in Case 3 the tendency of each sphere was to cause the supporting spheres to move outwards and rise above those of the next layer before any readjustment took place.

The space in the heaps not occupied by the spheres—the voids—was much greater in Case 1 than Cases 2 and 3; and greater in Case 2 than in Case 3. In fact, Case 3 gave the maximum density with minimum voids and the most advantageous use of the cementitious material filling the voids.

The voids in the three cases were as follows:—

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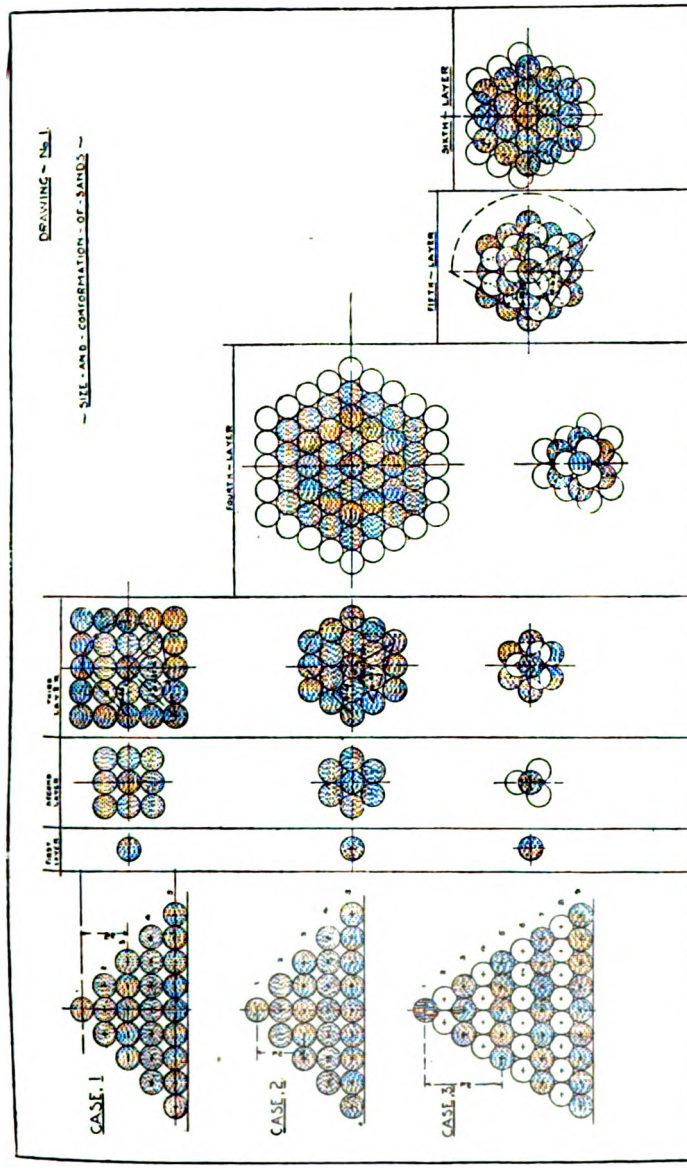

SIZE AND CONFORMATION OF SANDS: DRAWING NO. 1.

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	Solid Material.	Voids.	Loose Volume.
Case 1 . .	52·36	47·64	100·0
Case 2 . .	60·40	39·60	100·0
Case 3 . .	74·00	26·00	100·0

If it were assumed that the cement used to fill the interstices between the spheres was of the same degree of compactness as the heap of spheres, a portion only of the voids would be filled with solid cement. The proportions would then be as follows :—

	Solid Material. Per cent.	Solid Cement. Per cent.	Voids. Per cent.	Ratio of loose Volume to loose Cement.
Case 1	52·3	25·0	22·7	$\frac{100·0}{47·64} = \frac{2·1}{1}$
Case 2	60·4	24·0	15·6	$\frac{100·0}{39·6} = \frac{2·53}{1}$
Case 3	74·0	19·2	6·8	$\frac{100·0}{26·0} = \frac{3·83}{1}$

It might be taken for granted that even distribution of cement particles around the sand particles would give a stronger concrete than irregular coating. It was interesting, therefore, to see approximately how many layers of cement on each sand particle could be obtained with the cement required to fill the voids.

Let d = the diameter of spheres or sand particles,
r = ratio of diameter of cement particles to diameter of sand particles,
then rd = diameter of cement particles.

To obtain a continuous layer of cement, one cement particle deep, around each sand particle, the ratio of solid volume sand to solid volume cement would be $\frac{1}{3·14r}$. If the degree of looseness of the cement were taken to be same as that of the sand, the ratio of loose volume of sand to loose volume of cement for all three cases would be the same, viz., $\frac{1}{3·14r}$. If the loose volume cement were taken as in Case 1, the ratio of loose volume sand to loose volume cement would have the following values :—

Sand as in	$\frac{\text{Loose Vol. Sand}}{\text{Loose Vol. Cement}}$
Case 1	$\frac{1}{3·14r} = ·319$
Case 2	$\frac{1}{3·62r} = ·277$
Case 3	$\frac{1}{4·44r} = ·225$

Manifestly if r were small, either by the sand particles being large or the cement particles small, the volume of cement required was correspondingly smaller.

These theoretical considerations showed the great advantage of assembling sand particles in Case 3 over Cases 1 and 2, and bore out the well-known practical results, viz. :

(*a*) Finely-ground cement was advantageous.

(*b*) Coarse-grained sand was preferable to fine-grained sand.

(*c*) Ramming of dry concrete, which caused the particles to assume more nearly the conditions in Case 3 was advantageous.

(*d*) Regularity in size of the particles, i.e., proper grading, was essential.

Besides these generally recognised results, they indicated that the word "grading" must take on a new meaning. To obtain the densest, and therefore strongest, concrete a particular size cement grain should have a particular size sand particle. Likewise a particular size sand should have a definite size aggregate. With cement at $\frac{1}{200}$ in. sand would be $\frac{1}{15}$ in. and aggregate $\frac{1}{2}$ inch.

After a detailed account of the experimental work he had carried out, the lecturer said he had come to the following conclusions :—

(*a*) The examination of the different samples of sand showed how very varied in size were the particles. For strength of concrete it was desirable to eliminate all fine material, as the covering power of the cement was inversely proportional to the size of the particle covered. With cement at Standard Specification fineness, the desirable diameter of sand particles was as follows :—

For 1 to 1 mortar $d = \frac{1}{200} \div ·319 = \frac{1}{62}$, say 60 mesh.

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For 2 to 1 mortar $d = \frac{1}{200} \div \cdot 159 = \frac{1}{32}$ say 30 mesh.

For 3 to 1 mortar $d = \frac{1}{200} \div \cdot 106 = \frac{1}{22 \cdot 2}$ say 20 mesh.

(*b*) The angle of repose for granular material was not independent of the size of the particles, but decreased with decrease in size of the particles. This deduction might have to be varied as the result of further experiments. The angle of repose appeared to depend upon the smoothness of the particles and upon the limiting conditions of the lowest layer.

(*c*) It was clear that some method of packing the aggregate so as to bring it approximately into the condition of Case 3 would be more productive of strong or dense concrete than the addition of more cement.

Examination of concretes produced under centrifugal pressure showed that this treatment of the wet mixture did not pack the aggregate, nor could it be expected to do, unless the specific gravity of the solid aggregate material were greater than that of solid cement. Even then it was possible that the cement might become segregated away from the sand, an undesired result.

Institution of Structural Engineers.

A LARGE and representative gathering of engineers, architects, builders, and public men was present at the Criterion Restaurant on March 16 on the occasion of the annual dinner of the Institution of Structural Engineers.

After the loyal toast had been honoured, Mr. H. J. DEANE proposed "His Majesty's Ministers," in reply to which the Rt. Hon. VISCOUNT PEEL (H. M. First Commissioner of Works) said in the old days the engineer and architect were one. As we could not nowadays produce individual supermen, it seemed to be the practice to produce institutions and corporations, in which were collected together structural engineers and architects in the hope that together they would be able to solve all the problems of building. The great difficulty to-day was not to know what was an expert—that was easy—but to know what expert to choose. Sometimes he would like a committee of super-experts in which he could place complete trust. The public were bewildered by the advice given to them by experts in engineering. The problem of St. Paul's was an instance. On the one side he found a set of experts saying the only method of preserving the building was to grout the columns, while another set of experts, with equal authority, said the only way was to rebuild them. How could one judge between these diverse views of eminent experts? Who would tell them which was the right course to follow? It was the same with Waterloo Bridge—one set of experts told him it was possible to reconstruct the bridge, while another set of experts said it was impossible to restore the existing bridge unless it were taken down stone by stone and another and uglier bridge, which would take several lines of tramways, built in its place. He had been brought up to understand that the good and the beautiful and the true were one, but he was quite sure that, unfortunately, between utility and beauty there was often a vast chasm. He would like to have a set of super-experts or assessors who would assess and value all those considerations of beauty and necessity and utility, and tell him when he ought to expect beauty and when he ought to give way to baser and more necessary utility. Only those should be allowed to destroy who were able to rebuild nobly. Their duty was to educate people in the beauty and history and the wonders of ancient monuments, and that was one of the objects of the Office of Works. In their new buildings they strived to avoid anything commonplace, and to set a higher standard of beauty in public buildings. The Office of Works were not arrogant enough to try to reproduce the feeling and sentiment and spirit of another age, but they were ready to arrest the hand of Time where beauty lingered.

"The Learned Professions" was proposed by SIR CHARLES T. RUTHEN (Director of Housing), and responded to by Mr. TRAVERS HUMPHREYS.

MR. WENTWORTH-SHIELDS proposed "The Guests," to which MAJOR J. H. BEITH and MR. H. W. G. LEVESON-GOWER responded.

"The Institution" was proposed by the REV. F. H. GILLINGHAM, and responded to by MAJOR JAMES PETRIE, President.

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Thames ballast	,,	10	6
Broken brick (¾ in.)	,,	11	6
Best British Portland Cement	per ton 58*s.* to	63	0
"Ferrocrete" Portland Cement	delivered London 10*s.* per ton extra		
"Super-Cement"	per ton 88*s.* to	93	0
"Lightning" Brand Aluminous Cement	,, 100/- f.o.r. works.		

BOARDING FOR SHUTTERING—

		Sawn. s. d.	Wrot. s. d.
1 in.	per square	23 6	27 6
1¼ in.	,,	29 6	33 6
1½ in.	,,	35 6	41 0

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in.	from £23 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £23 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—

		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	0
9/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds	,,	14	6
¼ in. Rounds	,,	15	6

Breeze Slabs per yd. super: 2 in., 1/11; 2½ in., 2/4; 3 in., 2/9; 4 in., 3/6.

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 : 2 : 4—

		s.	d.
Do. do. in foundation	per yard cube	48	6
Do. do. in columns	,, ,,	53	6
Do. do. in beams	,, ,,	53	6
Do. do. in floor slabs 4 in. thick	per yard super	5	8
Do. do. in floor slabs 6 in. thick	,, ,,	8	5
Do. do. in floor slabs 9 in. thick	,, ,,	12	3
Do. do. in walls 6 in. thick	,, ,,	8	8

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—

		s.	d.
From ¼ in. to ⅜ in.	per cwt.	25	0
,, 9/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2⅞ in.	,,	23	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1¼*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2¼*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.* (per bend per cwt.).

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.* (per bend per cwt.).

SHUTTERING—

		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	,, ,,	1	1½
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1½
Do. do. as last, in narrow widths	,, ,,	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Carpenters working on old shuttering, 1/10½; Labourers on building works, 1/4½; Men on mixers and hoists, 1/5½; Bar-benders, 1/5½.

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Prospective New Concrete Work.

ALLOA.—*Bridge.*—The T.C. proposes to erect a bridge across the Forth.

BARNSLEY.—*Houses.*—The T.C. has approached the M.H. for permission to erect 450 houses.

BELFAST.—*Bridge.*—The T.C. has approached the Ministry of Home Affairs for permission to borrow £44,500 for the construction of a bridge over the railway at Tate's Avenue.

BELFAST.—*Houses.*—The T.C. is negotiating for a loan of £500,000 for housing purposes.

BEXLEY.—*Houses.*—The U.D.C. proposes to erect 100 houses.

BLYTH.—*Bridge.*—A scheme is being considered for the erection of a bridge, estimated to cost £80,000, to connect Blyth and Bedlington.

BRIDGWATER.—*Houses.*—The T.C. proposes to erect 30 houses.

BURNLEY. — *Concrete Road.* — The U.D.C. has applied for sanction to borrow £1,810, for the reconstruction of Albert Road and Fernlea Avenue in reinforced concrete.

CAMBUSKENNETH.—*Concrete Bridge.*—A scheme is in hand for the construction of a reinforced concrete bridge, estimated to cost £9,000.

CHERTSEY. — *Concrete Bridge.* — The R.D.C. is considering the construction of a reinforced concrete footbridge across the River Wey near Church Walk.

CHESTER. — *Road.* — The Corporation has approached the M.T. for a grant of £12,800 for the construction of the section of the ring road between Green Lane and Lache Lane.

CIRENCESTER.—*Road.*—A loan of £2,500 has been asked for by the U.D.C. for the construction of a road from Grove Lane to Gloucester Road.

COLCHESTER.—*Houses.*—The T.C. has applied for permission to borrow £26,967 for the erection of 53 houses.

COLWYN BAY.—*Houses.*—The U.D.C. has applied for a loan of £99,404 for the erection of 169 houses.

CROMER.—*Sea Defences.*—A scheme for the construction of sea defence works on the Cromer to Mundesley Road is being considered.

CYFARTHFA.—*Road.*—The Merthyr Corporation is negotiating for the purchase of land in connection with the construction of a proposed road through Cyfarthfa Works.

DERBY.—*Concrete Houses.*—The Corporation is considering the question of erecting concrete houses.

DOVER.—*Houses.*—The Corporation is considering the construction of 200 houses.

(*Continued on next page.*)

Concrete Memoranda.

IN response to several requests from readers we give below the weights of various materials used in reinforced concrete, and propose to make this a regular feature complementary to the Data on the previous page. The figures in all cases are average figures, and may vary in different cases.

Cement.—1 cu. ft. of Portland cement is calculated as weighing 90 lbs.; when filled very loose it may weigh as little as 75 lbs. per cu. ft.; if packed tight it will weigh 110 lbs. per cu. ft.

Water.—1 cu. ft. of water weighs 62½ lbs.; 1 gallon = 10 lbs.

Sand and Aggregate.—1 ton = 21 cu. ft. river sand; 22 cu. ft. pit sand; 22 cu. ft. ballast; 23 cu. ft. coarse gravel; 24 cu. ft. clean shingle; 40–50 cu. ft. broken brick.

Reinforcement.—Weight of square and round rods per ft.: ¼-in. sq. rods = 0·22 lb.; ¼-in. dia. round rods = 0·17 lb.; ½-in. sq. = 0·86 lb.; ½-in. round = 0·67 lb.; ¾-in. sq. = 1·92 lb.; ¾-in. round = 1·51 lb.; 1-in. sq. = 3·42 lbs.; 1-in. round = 2·62 lbs.; 1¼-in. sq. = 5·34 lbs.; 1¼-in. round = 4·19 lbs.; 1½-in. sq. = 7·69 lbs.; 1½-in. round = 6·04 lbs.

Concrete.—Weight of 1 : 2 : 4 concrete per cu. ft.: with coke breeze aggregate, 100 lbs.; with broken brick aggregate, 125–130 lbs.; with limestone aggregate, 135 lbs.; with shingle aggregate, 145 lbs.

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DOWNPATRICK.—*Concrete Road.*—The Down C.C. has authorised the Surveyor to reconstruct 550 yds. of the Lisburn-Lurgan Road in reinforced concrete.

DUDLEY. — *Houses.* — The T.C. has adopted a programme for the construction of 1,000 houses during the next two years.

DUNDEE.—*Concrete Houses.*—The T.C. is constructing new streets on sites to be used for the construction of 200 concrete houses.

EASTBOURNE.—*Concrete Houses.*—The Corporation proposes to erect 200 concrete houses.

GATESHEAD.—*Houses.*—The T.C. has applied for sanction to erect 140 houses, at a cost of about £70,094.

GOSPORT. — *Concrete Houses.* — The U.D.C. proposes to build 54 concrete houses.

GREAT YARMOUTH.—*Defence Works.*—The Corporation proposes to spend £1,000 on sea defence works.

GREENOCK. — *Concrete Houses.* — The Corporation proposes to erect about 400 concrete houses.

HERSHAM. — *Houses.* — The Walton R.D.C. has been asked to erect 50 houses in Hersham without delay.

HUDDERSFIELD.—*Houses.*—A housing scheme prepared by the Corporation includes the erection of 34 houses on the Grove Place site, and 17 on the Deighton Road estate.

HULL.—*Concrete Houses.*—The Corporation Housing Committee recommends the construction of 200 concrete houses.

JARROW.—*Houses.*—The Corporation has applied to the M.H. for sanction to erect 200 houses.

LEEDS. — *Houses.* — The Corporation has asked for sanction to erect 500 houses.

LLANDUDNO.—*Sea Wall.*—The U.D.C. has applied for a loan towards the cost of erecting a sea wall and promenade on the West Shore, which is estimated to cost £2,000.

MANCHESTER.—*Houses.*—The Corporation has passed a scheme for the erection of 10,000 houses during the next five years.

MANCHESTER.—*Sewage Works.*—The Corporation proposes to extend the sewage works at a cost of £480,000.

NEATH.—*Road.*—The T.C. is considering a scheme for laying a road, estimated to cost £84,000, from Cimba to Avon Valley Road.

NUNEATON.—*Houses.*—The T.C. has instructed the Borough Surveyor to prepare plans for the erection of 50 houses.

OSMOTHERLEY.—*Bridge.*—The Northallerton U.D.C. proposes to erect a bridge at the Bog Hole at a cost of £900.

POOLE.—*Groynes, etc.*—The T.C. proposes to construct concrete groynes and buttresses to protect private property on the shore at Sandbanks.

REDCAR. — *Houses.* — The Corporation has applied for permission to borrow £40,000 in connection with the erection of 100 houses.

ROSSINGTON.—*Reservoir.*—The Water Board proposes to erect a 250,000-gallon water tower at Rossington, and a 500,000-gallon reservoir at Conisborough Parks.

ROTHERHAM. — *Houses.* — The T.C.'s housing scheme provides for the erection of 700 houses during the next three years.

SHREWSBURY.—*Houses.*—The Housing Committee recommends the erection of 300 houses on the Racecourse site.

SIDCUP (KENT).—*Concrete Cottages.*—The District Council proposes to erect 100 concrete cottages.

SOUTH SHIELDS.—*Houses.*—The T.C. has applied to the M.H. for sanction to borrow £527,560 for the construction of 566 houses.

STOKE-ON-TRENT.—*Houses.*—The Corporation Housing Committee proposes to erect 700 houses.

STOKE-ON-TRENT.—*Sewage Works.*—The Corporation has applied for permission to borrow £19,618 for the purchase of land, etc., for the site of new sewage works at Trent Vale.

STRATFORD. — *Concrete Bridge.* — The T.C. proposes to construct a reinforced concrete bridge in place of Clopton Bridge.

WAKEFIELD. — *Sewage Works.* — The City Council has applied for a loan of £35,180 for extensions to the sewage works.

WARRINGTON.—*Concrete Houses.*—The T.C. has instructed the Borough Surveyor to prepare a scheme for the erection of concrete houses.

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Tenders Accepted.

ABERDEEN. — *Concrete Sewer.* — The T.C. has accepted the tender of Mr. W. Tawse, Torry, for the reconstruction of the reinforced concrete sewer at the North Esplanade West.

ACTON.—*Artificial Stone.*—The T.C. has accepted the tender of Mr. H. Morecroft, at £1,795, for artificial stone paving in The Avenue, Bedford Park.

AYR.—*Reservoir.*—The C.C. has accepted the tender of Messrs. McKean & Redwick, Glasgow, at £9,920, for the construction of a 600,000-gallon reinforced concrete reservoir, and laying about 2,300 yds. of 12-in. and 14-in cast-iron conduit.

CROYDON.—*Concrete Houses.*—The T.C. has accepted the tender of Messrs. John Laing & Son, at £930, for the erection of two concrete houses.

DEWSBURY.—*Sewage Works.*—The Corporation has accepted the tender of Messrs. Wellerman Bros., Ltd., at £18,564, for the construction of sewage disposal works.

FOLKESTONE. — *Concrete Sewer.* — The Corporation has accepted the tender of Messrs. J. Giel (Contractors), Ltd., for the construction of about 280 yds. of concrete sewer.

GOOLE.—*Water Tower.*—Messrs. Arnold & Son, Doncaster, have secured the contract for the construction of a 750,000-gallon reinforced concrete water tower for the T.C.

GUILDFORD. — *Retaining Wall.* — The T.C. has accepted the tender of Messrs. W. Stevens & Sons, Guildford, at £1,240, for the construction of a concrete wall at the depot.

HENDON.—*Sewer, etc.*—The U.D.C. has accepted the tender of Mr. J. Russell, Ilford, at £14,034 19*s.* 3*d.* for the construction of sewers, etc., which includes the laying of about 2,000 lineal yds. of 30-in., 36-in., and 42-in. concrete tubes.

LIVERPOOL. — *Concrete Houses.* — The Corporation has accepted the tender of Messrs. Wilson, Lovatt & Sons, Ltd., for the erection of 1,000 concrete houses on the "Boswell" system of construction.

LIVERPOOL.—*Reservoir.*—The T.C. has accepted the tender of Messrs. Peter Lind & Co., at £13,519, for the construction of a reinforced concrete reservoir at Bushbury Hill.

MALTON.—*Concrete Bridge.*—The East Riding C.C. has accepted the tender of the Yorkshire Hennebique Contracting Co., Ltd., at £1,215, for the construction of a reinforced concrete bridge at Malton.

MARTLEY.—*Retaining Wall.*—The R.D.C. has accepted the tender of Messrs. T. Vales & Son, Ltd., Kidderminster, for the construction of a concrete retaining wall.

NEASDEN. — *Concrete Houses.* — The Metropolitan Railway Co. have accepted a tender for the erection of 50 concrete houses.

NEWCASTLE - UPON - TYNE. — *Concrete Houses.*—The T.C. has accepted the tender of Mr. J. S. Hetherington for the erection of 300 concrete houses on the High Heaton estate.

PORT TALBOT.—*Concrete Jetties.*—The Great Western Railway Co. has accepted the tender of Messrs. Saxild & Partners for the construction of reinforced concrete jetties at Port Talbot Docks.

PRESTON.—*Concrete Houses.*—The T.C. has accepted the tender of the Condual Construction Co., Ltd., at £91,691, for the erection of 202 concrete houses.

ROTHERHAM. — *Sewage Works.* — The R.D.C. has accepted the tender of Messrs. Mollekin & Co., Ltd., Maltby, at £8,559, for the construction of sewage disposal works.

SOUTH PETHERTON (SOM.).—*Settling Tank.*—The Yeovil R.D.C. has accepted the tender of Mr. T. G. Walter, South Petherton, at £297 2*s.* 6*d.*, for the construction of a small concrete settling tank at the sewage disposal works.

SOUTH SHIELDS.—*Tea-rooms, etc.*—The T.C. has accepted the tender of Messrs. G. Bailey, Ltd., South Shields, at £6,922 13*s.* 6*d.*, for the erection of concrete tea-rooms and shelter on the North Foreshore.

TORQUAY.—*Pier.*—The T.C. has accepted the tender of Mr. R. E. Marracott, at £1,196 16*s.*, for the reconstruction of Haldon pier in reinforced concrete.

TORQUAY.—*Raft.*—The Corporation has accepted the tender of Messrs. Pearson & Co., Ltd., at £4,188 10*s.* 4*d.*, for the construction of a reinforced concrete raft and buildings for the extensions of the Newton Abbot Power Station.

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WELWYN.—*Concrete Factory.*—Messrs. F. D. Huntington, Ltd., have secured the contract for the erection of a reinforced concrete factory, 250 ft. long and 100 ft. wide, for the Shredded Wheat Co., Ltd.

WESTHAMPNETT (Sussex). — *Concrete Bungalows.*—The R.D.C. has accepted the tender of Messrs. W. J. Hart & Co., Ltd., Selsey, at £3,180, for the erection of eight concrete bungalows.

Recent Patent Applications.

17,574.—T. Brandwood: Construction of Concrete buildings.

210,760.—A. Petelot: Manufacture of Cement.

210,806.—H. A. Orlopp and D. B. Smith: Concrete-form tie and spacer.

211,497.—R. Decolland: Manufacture of aluminous cement.

222,426.—Soc. Anon. Des Chaux et Ciments De Larfarge Et Du Teil. Manufacture of fused cement.

222,427.—Soc. Anon. Des Chaux et Ciments De Larfarge Et Du Teil. Apparatus for the manufacture of cement by fusion.

224,214.—Soc. D'Etude Des Agglomeres: Refractory cement.

225,290.—A. E. White (Raymond Concrete Pile Co.): Apparatus for drying and heating stone, slag, and other materials.

225,613.—J. S. Hancock: Asbestos-cement sheets.

225,912.—J. R. Garrow, G. O. Case, and Novocretes, Ltd.: Mineralising fibrous materials, particularly sawdust and fibre, and cement.

226,037.—W. J. Jamison: Moulds.

226,348.—D. Cameron: Cements.

226,409.—J. T. Phillips: Casting concrete buildings *in situ.*

226,447.—K. Piehler: Concrete mixer.

226,885.—E. R. Calthrop: Reinforced-concrete structures.

227,522.—J. H. Walker and R. E. Knight: Reinforcements.

227,704.—H. W. Evans: Machines for making concrete blocks.

227,724.—R. C. Stubbs: Mixing concrete under pressure.

227,736.—T. Wood; Concrete buildings.

227,969.—F. Nitzsche: Reinforced concrete foot for wooden poles.

228,346.—H. Weston: Reinforcements.

228,438.—J. Smeets: Machines for moulding concrete blocks, etc.

228,485.—A. E. White: (Raymond Concrete Pile Co. Inc.). ; Concrete piles.

228,580 and 228,826.—E. G. R. Marks (Concrete Tile Machinery Co.): Tile-making machines.

229,149.—C. F. Bamford: Building blocks.

New Companies Registered.

BRITANNIA CEMENT CO., LTD. (203239). Registered January 20. Britannia Cement Works, Northfleet. Cement manufacturers. Nominal capital, £15,000 in £1 shares. Directors: J. S. Killick, Stone Park, Greenhithe; F. M. Bond, 41, Finchley Road, N.W.8; G. Mitchell, 24, Corfton Road, W.5.

CALWAY CONSTRUCTIONS, LTD. (202869). Registered January 3. Specialists in concrete construction, Cwm Road, Upper Strand, Swansea. Nominal capital, £5,000 in 1,000 ordinary shares of £1 and 80,000 deferred ordinary shares of 1s. each.

RED-MARL BLOCK CO., LTD. (203056). Registered January 12. 129, Cemetery Road, Sheffield. Manufacturers of concrete blocks, etc. Nominal capital, £2,000 in £1 shares. Directors: H. Bartholomew, 61, Headland Road, Crookes, Sheffield; H. B. Scaife, 14, Damer Street, Sheffield.

Trade Notices.

A CATALOGUE of industrial sands has been issued by Mr. A. L. Curtis, of Westmoor Laboratory, Chatteris, included in which are many sands suitable for concrete work; tensile and compression strengths of these sands are given in the booklet.

MESSRS. DAVID WESTON & CO., Ltd., have issued another of their "King's Highway" series of booklets dealing with modern road construction. This issue deals with reinforcement for roads and foundations with "Concretus" double-layer reinforcement.

THE address of the branch office in Scotland of the British Reinforced Concrete Engineering Co., Ltd., is now 144, St. Vincent Street, Glasgow, C.2. Telephone: Central 2726.

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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EDITORIAL NOTES.

Concrete and Architecture.

IT has always been difficult to understand the feeling of hesitation, amounting in some cases to a frankly expressed preconceived dislike, which most architects display when discussing the use of reinforced concrete for building anything except a foundation or a retaining wall. Where the problem of making the best of the new material has been honestly approached, such as in some of the newer factory buildings in this country and more frequently in all kinds of monumental structures abroad, some excellent architectural conceptions have been evolved, but we venture to think that no good design can ever be produced with a material which is not fully understood and which is handled without a clear knowledge of its possibilities and an honest endeavour, free from prejudice, to adapt it to the best architectural effect. Would we have had our glorious monuments of the past in stone if stone had been looked on with suspicion and distrust? These

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old buildings give us pleasure because their builders seized enthusiastically upon new forms applicable to the material as they were discovered, and took a delight in seeing what they could do with them, and it is not until this spirit is shown in the use of reinforced concrete that the best results will be produced and concrete take its place with stone and brick as a medium of architectural expression as distinct from a mere building material. It is perhaps unfortunate that the development of reinforced concrete coincides with a universal demand for cheapness, resulting in the utmost economy of materials which gives the thin appearance to the exterior columns and beams of so many reinforced concrete buildings. It is this slenderness of construction which upsets all architectural notions, and which makes reinforced concrete design a radical departure from other materials; the more weight stone or brick has to carry so the thickness of the wall or column has to be increased in proportion, whereas with ferro-concrete the extra strengthening may be obtained by increasing the amount of steel reinforcement and not appreciably adding to the volume of the member. Is it necessary, however, when an architectural effect is required, always to reduce dimensions to the finest point possible consistent with the requisite strength? As Mr. Maxwell Ayrton, F.R.I.B.A., recently said: "Very few of us, if we are designing a stone column, calculate the exact area of the column and reduce it to the load it will safely carry. If we want a 2-ft. column we have it, considering it only from the point of view of design. It may only be necessary for structural strength for a reinforced concrete column to be 6 inches in diameter, but why should we sit down and imagine that in future we have to design spiders' webs. It is not fair to the material, and that feeling has arisen from the fact that up till now architects have used reinforced concrete merely as a frame to be covered up as quickly as possible, and so long as we do cover it up we shall get no farther forward."

The importance of obtaining the maximum floor space is fully realised, but in many cases it should be possible to use larger columns with a leaner concrete which while not adding to the cost would give the feeling of strength so much to be desired from the architectural point of view. In a reinforced concrete frame building there is no reason why the exterior piers should not be wider than is now so often the case, the extra width of the piers being taken out of the filling between them. Whether a definite and satisfactory architectural style for concrete will be evolved remains to be seen. Certainly it will be difficult, for the fog of precedent and past styles is made enormously difficult to get away from by the fact that reinforced concrete is applicable to any form of construction and may be used in place of any material.

The paper by Professor Beresford Pite at the Royal Institute of British Architects, given elsewhere in this issue, is a stimulating encouragement to architects to take more interest in reinforced concrete, and is to be welcomed as a sign that not all our architects are content to see the architecture of reinforced concrete left principally to specialists; but, as we have stated, the wonder is that they did not immediately grasp the opportunity of adopting and seeking the best results for a new material with such enormous possibilities which modern science has placed at their disposal. What would have been the attitude of architects towards concrete had it been as expensive as marble is an interesting line of thought, for its very cheapness is urged against it when the undesirability of using the same material for the façades as for the foundations is put forward.

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Undoubtedly the future of concrete for monumental building rests largely upon surface treatment, and it is to be hoped that when this field has been fully explored and satisfactory means found of giving concrete a pleasing, perhaps coloured, texture an analogy will be found between stone when it is first quarried and after it has been dressed and concrete before and after it has been treated.

Progress of Concrete Housing.

In the first five months of this year we have mentioned in this journal the definite acceptance of tenders for the erection of no less than 5680 concrete houses in various parts of the country, and in addition we have drawn attention to proposals which have not yet reached the stage of inviting tenders involving a further 7482 houses for which concrete is the material proposed. From an examination of all the housing tenders accepted during this period it appears probable that concrete is now the predominating material in use by local authorities for housing schemes, and unless conditions in the building industry alter so that conditions and prices are easier in regard to the labour and materials necessary for building in brick, which seems unlikely for some time to come, we shall have more houses of concrete than any other material. What measure of success the steel house will have remains to be seen, but the difficulties which are being experienced with labour for the erection of this type of house, and the fact that the cost of the steel house is more than the figure for which concrete houses are now being built in many districts, are factors which cannot be encouraging to the sponsors of this method of construction; neither does the claim that steel houses can be built in a fortnight carry much weight with those who realise that the time taken to build a house does not depend on the time required to erect the superstructure alone, for on a fair size scheme, organised so that building closely follows the work connected with foundations, roads, sewers, and other services, there should be hardly any difference in the period necessary for a given number of houses in either material. We do not decry any method or material which will provide cheaply and expeditiously the much-needed housing accommodation, but in view of the extent to which local authorities are now using a tried material in concrete it seems doubtful if they will adopt steel unless there is a substantial saving in first cost, which the prices quoted show is not available at present.

Prejudices in Cement Testing.

It has been difficult at all times to dissociate the testing of cement from prejudice and misconception. Some of the tests that were current twenty years ago had no scientific basis, and have therefore been discarded, yet they were regarded as reliable indications of quality by engineers of standing. The Engineering Standards Committee, by its issue of the British Standard Specification for Portland Cement in 1904 and by its frequent revisions of this specification, has done much to disperse prejudice and to apply scientific principles to the testing of cement.

Primarily, cement is tested for three purposes, viz. to ascertain its strength, qualities of endurance, and setting properties. In most cases the strength of cement is determined by tensile tests, its endurance is determined by the Le

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Chatelier soundness tests and by chemical analysis, and setting properties are revealed by observations on a block of cement gauged with water. Cement in practice is required to resist compression stresses and is seldom or never called upon to withstand tension. Hence it is not quite rational to judge of the strength of cement by tensile tests to the exclusion of compression tests. But tensile tests are more easily made than compression tests and require less expensive appliances, while it has hitherto been considered that there is a fairly definite ratio between the tensile and compressive strengths of cement. It has, in fact, been generally assumed that cement is ten times as strong in compression as in tension, so that if the tensile tests of any particular cement were satisfactory it was a foregone conclusion that the compression strength was satisfactory to the same degree. There has been a considerable body of evidence in support of this assumption, and so far as cement testing was concerned compression tests were regarded as unnecessary.

With the developments that have lately occurred in cement manufacture, however, there appears to be some doubt as to whether this close relation between the tensile and compression qualities of cement exists in connection with the brands of rapid-hardening cement that are becoming prominent.

The following examples will illustrate the point :—

3 PARTS STANDARD SAND TO 1 PART CEMENT (lbs. per sq. in.).

	24-HOUR TESTS.			7-DAY TESTS.		
	Tension.	Comp.	Ratio Comp./Tension.	Tension.	Comp.	Ratio Comp./Ten.
Cement A. . .	133	1125	8·4	397	3950	9·9
Cement B. . .	333	3425	10·3	730	10000	13·7
Cement C. . .	582	7150	12·3	680	9450	13·9

Cement "A" may be taken as an ordinary Portland cement and Cements "B" and "C" are examples of the rapid-hardening cements referred to in detail in our recent issues. From these figures it appears that the higher quality cements have a greater ratio between compression and tension than ordinary cement, and the true value of a cement may be misjudged if its tensile tests are taken as a criterion of its strength. The difference in the ratio is too great to be ignored, and it seems that in time the convenient tensile test will have to give place to the compression test as a more reliable indication of the value of the cement in actual use. The prejudice—hitherto well-founded—in favour of tensile tests must, in fact, give way to a test that is more rational when considered in connection with the application of cement.

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Architecture in Concrete.

Professor Beresford Pite on "A Concrete Style."

"THE door is opened. The designer is no longer in a stonework prison. The realisation of the possibility of the deliverance from ideal proportions established in masonry by the substitution of reinforced concrete is the taking of a very considerable step in the path of architectural progress. The dead hand of the past has been lifted from architecture; Renaissance is no longer its aim. We are marching to a dawn; but it is not of the long-set sun of Classical precedent—it is in the light of an entirely new science of building that glorious adventure is to be sought."

The above is an excerpt from an address given by Professor Beresford Pite, F.R.I.B.A., at a meeting of the Royal Institute of British Architects last month. It is not often we hear concrete so sympathetically discussed at a gathering of architects in this country, and the following remarks by such an authority as Professor Pite are of particular interest as showing the views of a scholarly

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Reinforced Concrete Flour Mill at Bethune.

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New Russian Church at Nice.

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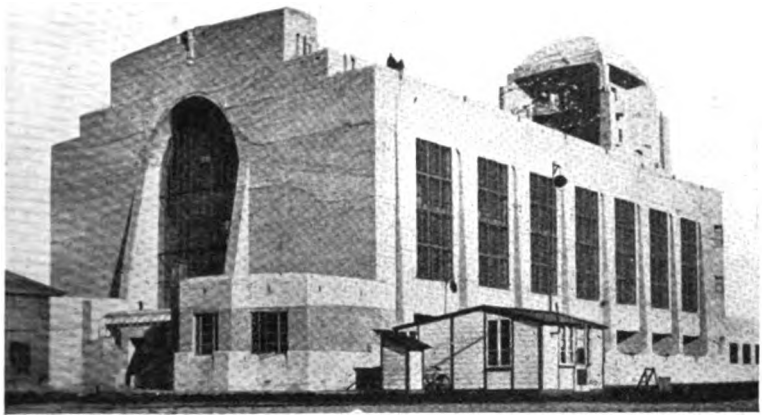

Reinforced Concrete Radio Station at Kootwijk.

architect on the subject of the architecture of reinforced concrete :—

The factories, public buildings and bridges which at present form the bulk of the work executed in reinforced concrete in this country illustrate at least some of the practical applications of the material, but on the Continent it is evident that a courageous architectural spirit has seized upon the possibilities, artistic as well as practical, of this novel substance of construction, and results that can only be described as revolutionary have appeared. The use of a homogeneous material, as our French brethren have perceived, is revolutionary rather than evolutionary, and raises manifold questions as to its due architectural expression ; and necessarily also it involves reconsideration of methods in the study of the practice of design with the courses of training leading to its fulfilment.

It is not the least useful of the qualities of the combination of steel and concrete that an essential unity of structure results from the mixture of minerals and metal. The advent of concealed steel framing into building used to be justified by the analogy of the human skeleton, and answered by the purists with the retort that the flesh had no structural relation to its bones. But the simile now holds good, and perhaps carried farther into the analogy of muscle and tendon which yield the beauty of form to the animal figure.

Ultimately, of course, the static result of the building is identical with that of the older forms of construction, though the architectural expression which depends so largely upon tradition is very different. The column and lintel of reinforced concrete still exercise the functions of those of the Doric order, but with greatly extended range. Imagination and sympathy with Greek thought should not be alarmed by the suggestion that the Parthenon, if constructed of reinforced concrete, would only need one supporting column at each angle of the peristyle, and that the subtleties of intercolumniation may be dispensed with. The cantilever can now be constructed and employed with a certainty that opens fresh possibilities of adventure beyond those so laboriously and wonderfully developed in Indian Dravidic architecture. As in bridge construction, it replaces the arch and dispenses with buttresses. Again, the emotions may be perplexed by a dream of the vaults of Rheims, Amiens or Beauvais carried aloft without the external scaffolding of flying buttresses, as graceful and necessary to our minds as the peristyles of Greece, but it is evidently not only possible but as proper to employ the resources of ferro-concrete for imaginative architecture as masonry. The arch in ferro-concrete alters its nature ; to the Greeks it was probably but a subterfuge ; to the Romans a constructional wonder known only in its semicircular form with its regular elliptical intersections, but developed in mass concrete with brick reinforcements with an energy and success

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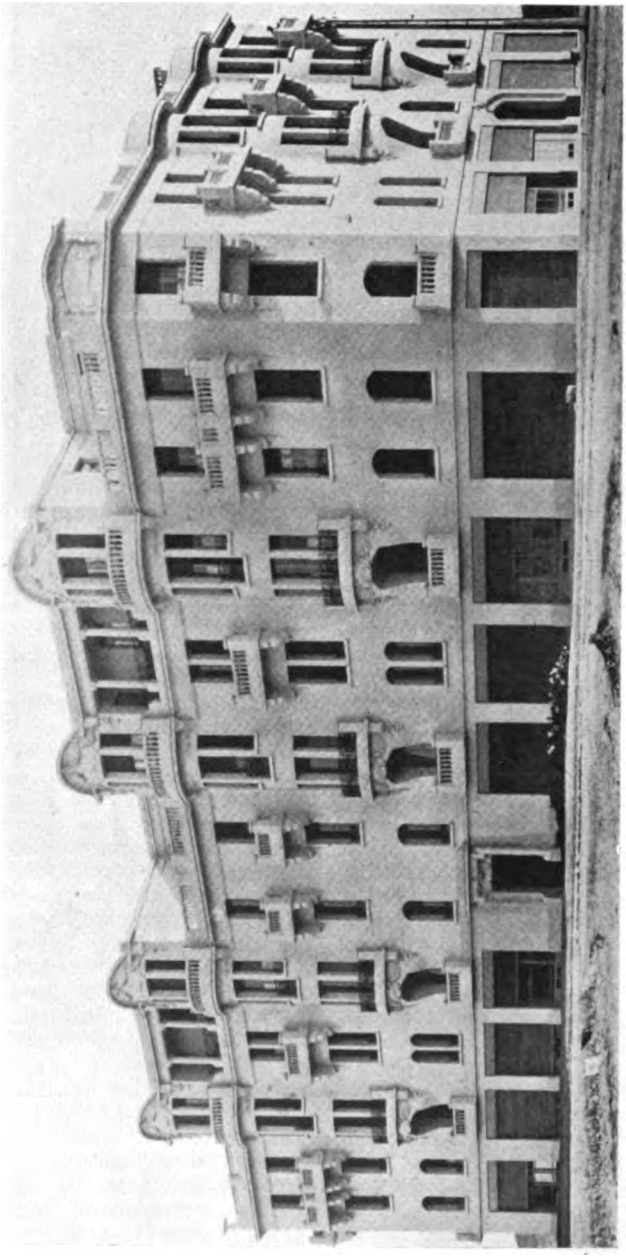

Reinforced Concrete Building at Casablanca for the Société Agricole du Maroc. [*Coignet Design.*

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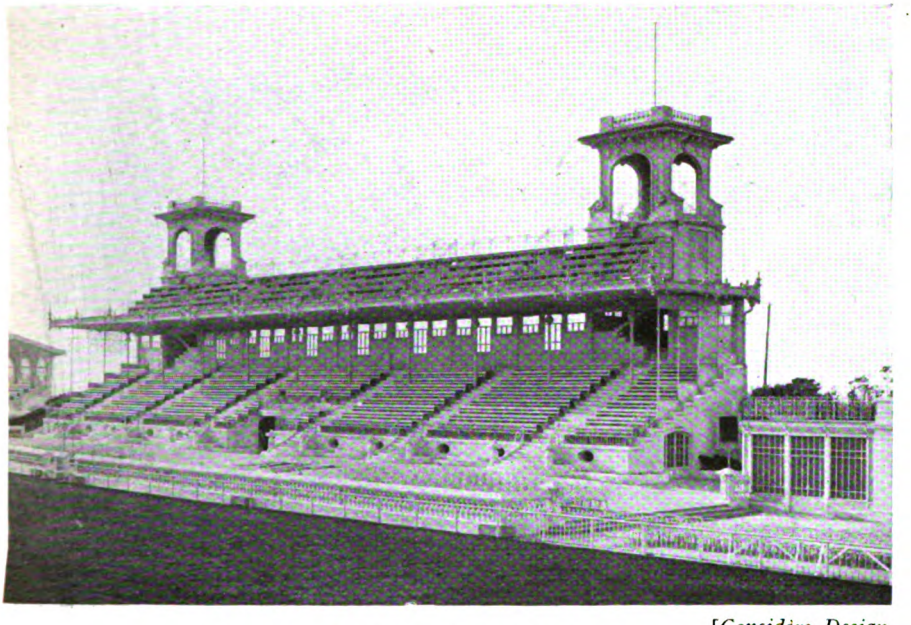

[*Considère Design.*

Reinforced Concrete Grand Stand at Nice.

that may be considered as one of the most potent illustrations of an empire's character supplied by architectural history. The Roman genius that had established a new architectural beauty in sheer construction effect, with proportions of strength and dignity undreamt of by the Greeks, can well be credited with revelations of the possibilities of ferro-concrete construction exceeding anything that even now has been attempted, either in the old world or in the new.

The accomplishment of considerable buildings in concrete without any resort to the paraphernalia of stonework architecture is a step forward that we should pause to recognise. It is to be realised that our civil and ecclesiastical architecture, heavily labouring in the endeavour to revivify the dead, cannot be reckoned as the most characteristic building expression of our age. It only suggests that architects minister to the prejudices of unperceptive clients—but in our midst, upon a scale far exceeding in volume houses, churches and municipal halls, commercial building is marking our epoch with the factories and storehouses of imperial industry and wealth. Reinforced concrete has already provided those whose part it is to contemplate the progress of the arts of building from the external standpoint of contemporary architecture with a new conception of simple masses, self-reliant and highly expressive of purpose.

A further element is the advent to architecture, through ferro-concrete, of an originality that is consequent to the unusual material. The long drawn out desire for a new style may thus find its fulfilment safely and reasonably in the novelty of the substance and method of building. This originality is necessarily natural and unforced. It lies not only in perceiving the architecturalness of big commercial buildings and in obviousness of purpose, but in the new conception of proportion that ferro-concrete involves. The factors of architectural safety are no longer resident in the classic tradition of a stonework "order," originally based on the possible bearing of a marble lintel, with its column, architrave and intercolumniation, or upon the apparent balancing of an arch. These have passed away; at all events the logic of their application to concrete building has ceased. We are now accustomed, or becoming so, to any length of span, and

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Reinforced Concrete Barracks at Casablanca. [*Coignet Design.*

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therefore the relation of voids to solid, once a doctrine of design, is freed from stonework precedents—the case is altered, and we must readjust our canons of design. It will be clear that if the conception of satisfactory proportion based upon stonework building is relative to precedent and not absolute, ferro-concrete must create its own standards. It is, in fact, doing so by every example that frankly employs its resources.

This realisation of the possibility of deliverance from ideal proportions established in masonry by the substitution of a new material is the taking of a very considerable step in the path of architectural progress. Within the sphere of those buildings to which on account of their size ferro-concrete is applicable, and these are the largest, the revolution of architectural ideals must involve a readjustment of ideals in design that is searchingly fundamental. A final positive gain to architecture in the advent of ferro-concrete is the new spirit of artistic and constructional adventure which the elasticity of the material and the novelty of its applications reveal. The designer is no longer in a stonework prison; the first freedom should be dedicated to exploring and exemplifying frankly the new characteristics of virility and adaptability which lie in the material. It will be a service demanding courage as well as clear-sightedness, but it is demanded in order to pave the way for the recognition and

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Reinforced Concrete Signal Box in France.

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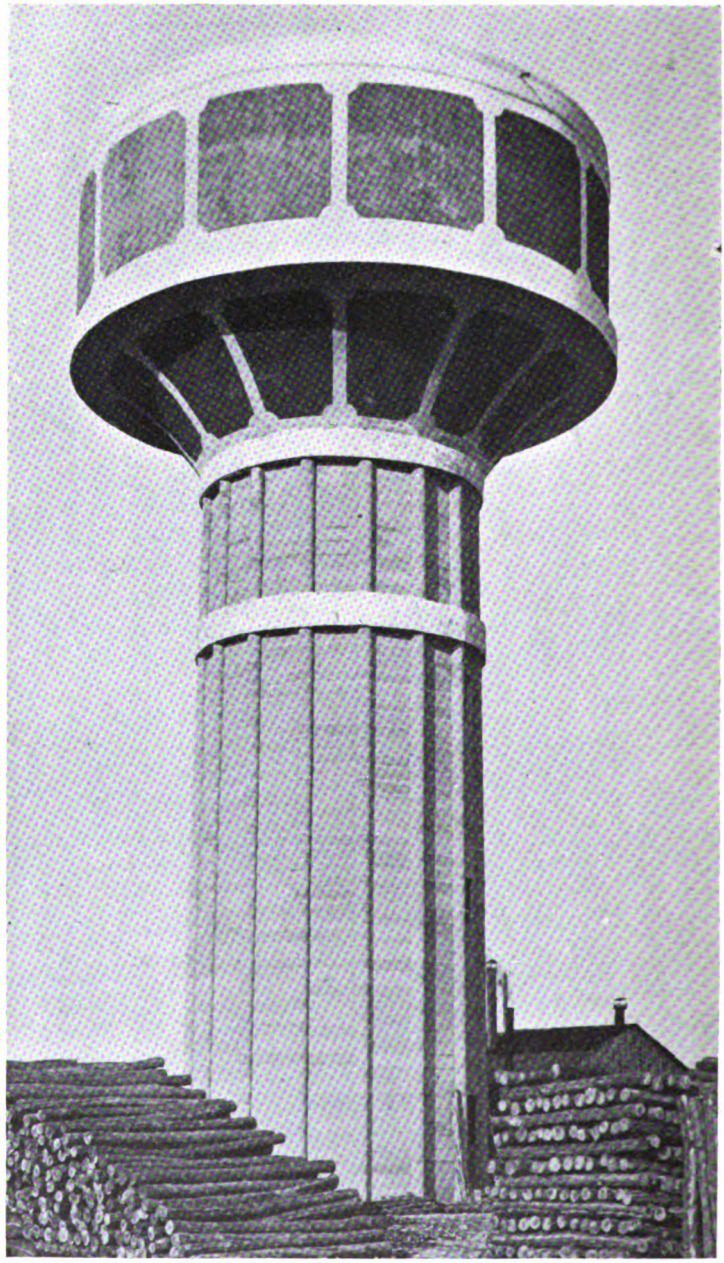

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Reinforced Concrete Water Tower at Lens, France.

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[*Sir John Burnet & Partners, Architects.*

Concrete Pavilion at Ramsgate.

establishment of the new conceptions of architectural proportion which the use of ferro-concrete has introduced.

The question of the ability of ferro-concrete to provide a satisfying architecture was not so obvious to answer as it might appear to be. We must leave on one side the provocative subject of historic style and insist on a clean slate whereon the new concrete style is to be planted. The qualities which characterise ferro-concrete building comprise a new largeness and freedom of scale, simplicity in directly expressing purpose, originality of treatment, a novel standard of sense of proportion, and the but partially explored possibilities of constructional adventure in design which are not attainable in materials that are wrought when solid. With such elements and potentialities the architect should recognise and perhaps emphasise the special qualities and purposes of the building and its construction; at all events, not dissemble them as if they were antagonistic to his ideals. He can bring organisation to bear upon the masses of the building by simple symmetrical arrangement, balancing and gradating projections both on plan and elevation, in due regard to the premiss of utility. Or by the emphasis of parts mould irregular masses into picturesque groups of contrasting forms, for by taking thought, intelligence, or idea, may be imparted to a chaos. Contrast between the horizontal and vertical direction of stories may be secured and marked by the arrangement and division of openings in the wall. Piers and beams may be expressed as they are necessitated by the construction without other artistic interference, the problems of proportion being left to the structural engineer. Such suggestions might be multiplied; they are but general; each building will furnish its own opportunity for design, but provided that intellectual interest is manifested it may be confidently asserted that artistic result is assured and the path leading to architectural success entered upon, though in a hitherto unexplored territory.

The crux is detail and ornament. A little figure modelling on a flat surface will go a long way. The treatment of surfaces suggests possibilities that may be at once interesting and economical, such as rural plastering, sgraffito, mosaic or ceramic inlays.

In the advent of ferro-concrete a new opportunity may be hailed for architecture if the leaders of the profession are at all conscious of the universality of their art.

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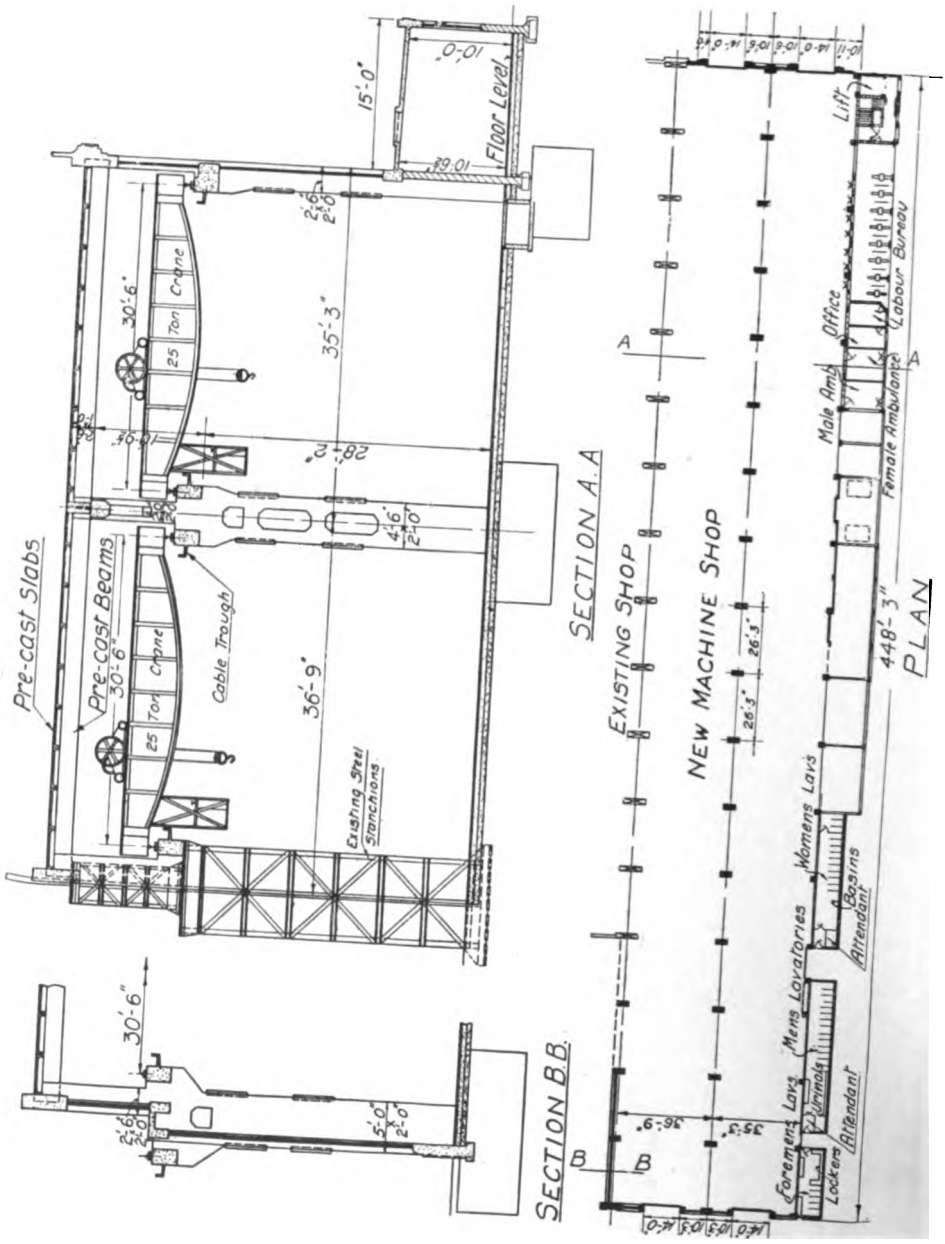

Reinforced Concrete Factory Building at Witton, Birmingham. [*Mr. Thomas Wallis, F.R.I.B.A., Architect.* (*See p.* 251.)

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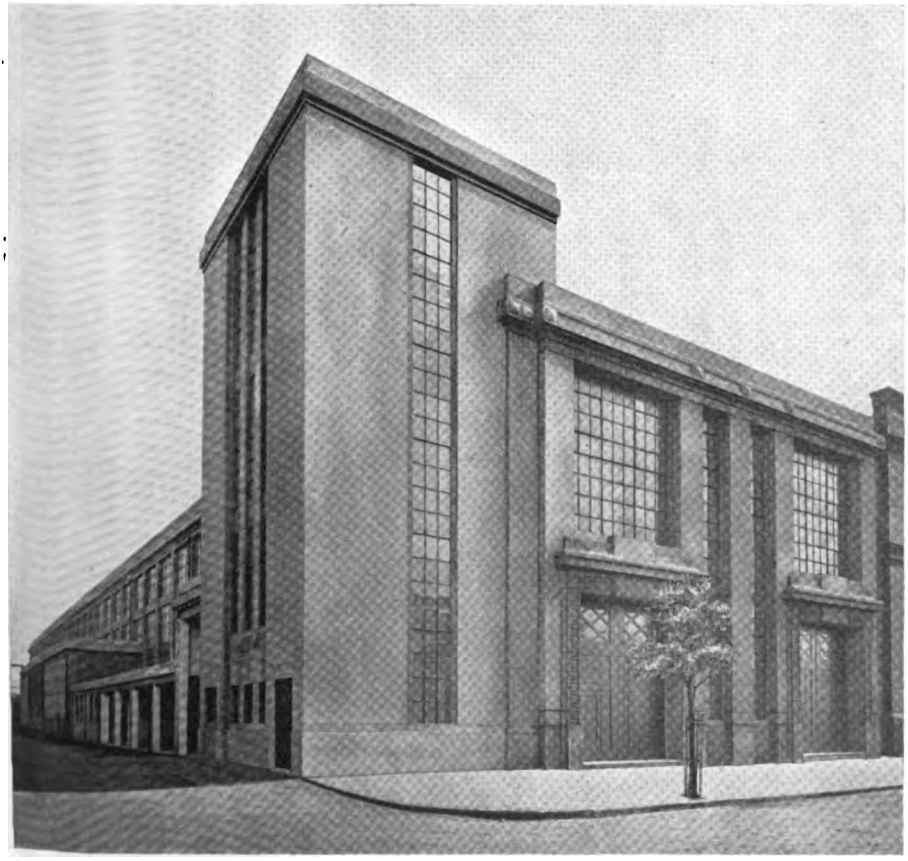
Reinforced Concrete Factory Buildings at Witton, Birmingham.

In the last century, when the erection of factories and dwellings for the workers changed the face of the country, little thought was given to architecture or the effect of the buildings on the district in which they were built. The result is the conglomeration of ugly and depressing factories and warehouses and slums which form most of our manufacturing centres to-day, and which are the despair of the architect, town-planner, and social reformer alike. There is, however, a growing appreciation of the value of attractive industrial buildings and garden cities; business firms are realising the advertising value of making their products in pleasant and hygienic buildings and surroundings; and, perhaps the most important factor of all, the architectural profession is making it known that the attractive appearance or otherwise of a structure depends chiefly upon proportion, and that a building which is good to look at need cost no more than one which is an eyesore.

The new extensions of the works of the General Electric Co., Ltd., at Witton, Birmingham, designed by Mr. Thomas Wallis, F.R.I.B.A., M.I.Struct.E., of Messrs. Wallis, Gilbert & Partners, Architects, of 29, Roland Gardens, London, S.W., are an excellent example of a modern architectural



[*Mr. Thomas Wallis, F.R.I.B.A., Architect.*

Reinforced Concrete Factory Building at Witton, Birmingham.

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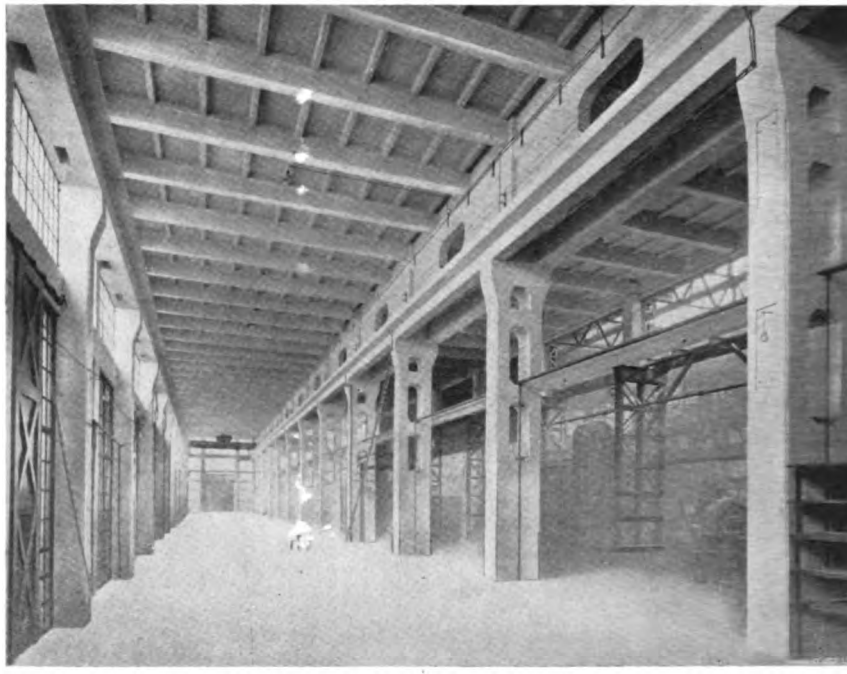

[*Mr. Thomas Wallis, F.R.I.B.A., Architect.*

Reinforced Concrete Factory Building at Witton, Birmingham.

factory building. The extension, the whole of which is in reinforced concrete, comprises an additional two bays to the machine shop, measuring 448 ft. 3 in. long, with a total width of 72 ft., and a height from floor to roof of 41 ft. 6 in., with smaller buildings at the side. The main building is at present only one story high, but the roof is designed for the strength of a floor so that in the future another roof may be constructed over it to convert the building into a two-story one. It will be seen from the plan that stairs are provided with space for a lift as a means of access to the future story.

Throughout the whole length of the two bays tracks are provided to carry electric overhead travelling cranes with a lifting capacity of 25 tons each. The columns of the main building are spaced at 26 ft. 3 in. centres to correspond with the existing steel stanchions. The centre columns rest on bases 14 ft. long by 6 ft. wide, and have a section of 4 ft. 6 in. from the ground floor to the crane floor. The columns are pierced by three holes, the two lower ones being 5 ft. 2 in. long by 2 ft. wide, and the top one 2 ft. by 2 ft.; the former are for economy and effect only, while the latter is for housing a line of shafting. Above the crane beams the columns diminish to a section of 2 ft. by 2 ft. The external columns rest on bases 9 ft. long by 6 ft. wide and have a section of 2 ft. 6 in. by 2 ft. up to crane level and a section of 1 ft. by 3 in. up to the roof. The internal faces of all the columns have provision made on them for attaching special slotted inserts to which countershafting or any other mechanical equipment may be fastened by bolts without any cutting into finished concrete. The crane beams, resting on brackets on the columns, vary in size according to the loads they have to carry; those in the external bay are 30 in. deep by 15 in. wide, being designed to carry a 25-ton travelling crane, and those in the internal bay are 30 in. deep by 18 in. wide to carry two 25-ton travelling cranes operating together. The crane beam adjoining the existing shop rests on existing steel stanchions, the top portions of which are encased in concrete as shown

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on Section AA, the concrete providing a good means of anchoring the beam by means of reinforcing bars. The travelling cranes run on 56-lb. rails secured by ⅝-in. coach screws to 10-in. by 4-in. timber bearers which in turn are fixed to the crane beams by ⅞-in. ragbolts. Concrete troughs are formed on the sides of the crane beams to carry power and lighting cables. The bottom of the trough was cast simultaneously with the beams and reinforced by bars projecting from them, the sides being formed by attaching "Hy-Rib" to the reinforcing bars and coating it with cement plaster, thus eliminating the use of centering.

The roof, which is designed to carry a future floor load, was not cast *in situ* like the columns and crane beams below but was made in units on the ground, and after the concrete had matured was hoisted into position in the manner of structural steelwork. This method was considered more economical than casting it in position, as owing to the great height of the roof an enormous amount of propping and staging would have been required to support the false-work while the concrete was being placed. The three lines of main beams, shown in Section AA, were cast *in situ*, as the centering for these could readily be carried by the crane beams below. Pre-cast secondary beams spaced at 8 ft. 9 in. centres carry the roof in two spans. These beams, 21 in. deep and 12 in. wide, were designed strong enough to carry their own weight and the slabs that rest upon them. The pre-cast slabs are 8 ft. 1 in. long, 4 ft. wide and 2½ in. thick, with a beam 9 in. deep on all four edges, and have the appearance of shallow bottomless boxes. They were placed on top of the beams, the edges of the slabs taking a bearing on each side of the beams, leaving a space between the slabs and over the beams 8 in. wide and 9 in. high. This space was afterwards filled with concrete reinforced with bars secured by stirrups projecting from the unit beams for that purpose, thus making the finished beams 30 in. deep, and thereby adding considerably to their strength. This method of construction has proved quite successful in this instance, the hoisting of the units presenting no great difficulties, the weight of the beams being not more than 4 tons and the slabs only 12 cwts.

The elevations to this building, even in their present incomplete form, have a very pleasing effect, and the modelled concrete cornice at the present roof level adds considerably to the impression conveyed by the whole. All available space is devoted to window area, the maximum amount of natural light thus being obtained. A horizontal band of concrete runs around these two elevations at the same level as the crane beams, and serves the dual purpose of concealing the crane beams and effectively relieving the monotonous appearance of a large expanse of very long sashes. The tower encloses the staircase and lift, and will be surmounted by a cupola when the second story is added. The two doorways, surrounded by architraves all cast in concrete, are striking features of this elevation. The reinforced concrete was designed by the Trussed Concrete Steel Co., Ltd., South Kensington, and executed by Messrs. W. J. Whittall & Son, of Birmingham.

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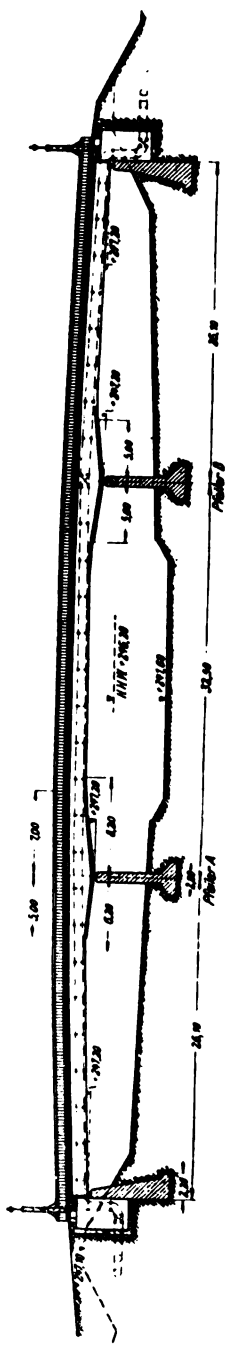

Fig. 1.

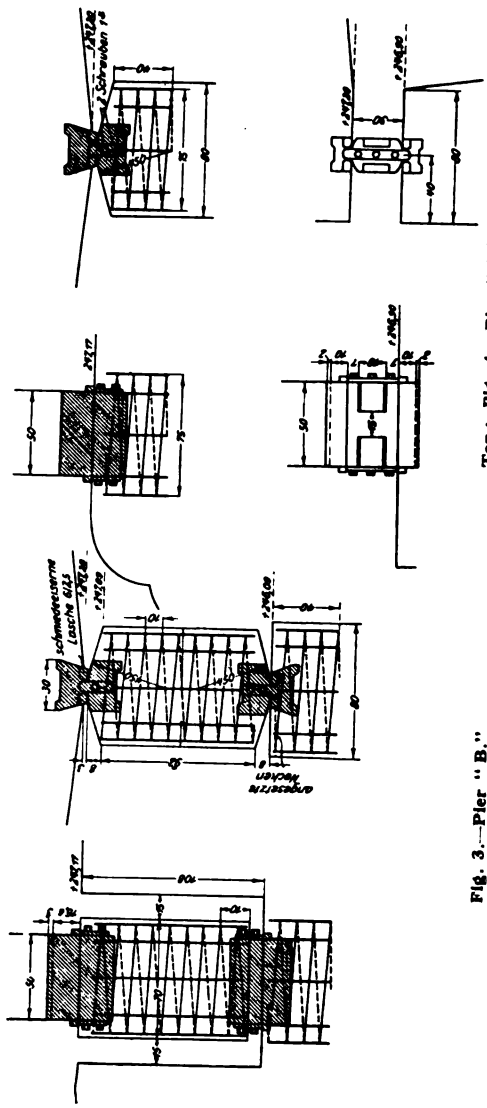

Fig. 3.—Pier "B."

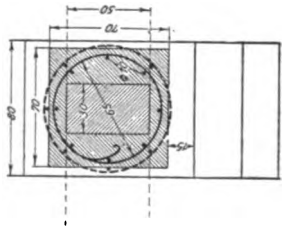

Fig. 6.

Top: Fig. 4.—Pier "A."
Bottom: Fig. 5.—Bearing at Abutment.

A Long-Span Beam Bridge.

(*See p.* 255.)

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Long-Span Beam Bridge.

In a recent number of *Beton u. Eisen* a description was given of a reinforced concrete bridge at Pforzheim which has some interesting features.

The main reason for the construction of this bridge, which has a centre span of 115 ft. and two side spans of 86 ft. each, was to carry the two new 19½-in. diameter mains of the town water-supply from the pumps on the right bank of the river Enz to the other bank. Provision was also made for pedestrian traffic. The water-mains are situated between the main girders of the bridge and are supported on reinforced concrete cross-girders. The pipes are covered by a decking extending from one main girder to its neighbour, but in the vicinity of the piers this decking is not monolithic with the main girders. At these points the decking consists of separate pre-cast reinforced concrete slabs giving a means of access to the pipes for maintenance purposes (see *Fig.* 2).

For distances of about 20 ft. for the shorter spans, and 27 ft. for the middle span, from the river piers the main girders are connected on their undersides by a reinforced concrete compression flange which helps to resist the stresses caused by the negative moments at the piers. Thus the cross-section of the bridge in the neighbourhood of the piers is a closed box shape. Allowance was made for the variable moment of inertia of the girder.

Two piers only were constructed in the river, each being 31½ in. thick, and the normal high-water cross-sectional area of flow was thus diminished by only 3 per cent. On account of the great lengths of the span considerable alterations in the girder lengths were to be expected due to expansion and

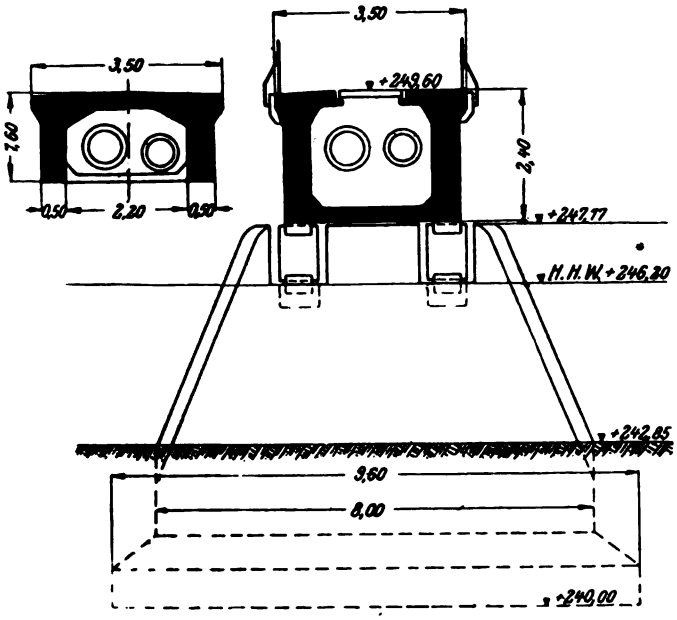

Fig. 2.

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contraction. The supports of the girders were therefore designed so that any alteration of girder length might take place without constraint. Originally it was intended to use cast-steel bearings similar to those used in steel bridge practice, but as these could not be obtained without considerable delay a combination of cast-iron and reinforced concrete was substituted at the piers and cast-iron alone at both abutments. The bearings used are shown in the illustrations reproduced from our contemporary. A pressure of 42,700 lbs. per sq. in. was allowed between cast-iron and cast-iron and the radius of the surface of contact was computed by the Hertz-Liebbraud formula. The reinforced concrete portion of pier " B " was constructed with a high-strength cement and spiral binding in order to give a resistance to compression of 1,440 lbs. per sq. in.—W.S.G.

Questions and Answers relating to Reinforced Concrete.

Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.—Ed.

Concrete Floors at Wembley.

Question.—I have to concrete a large area to form a floor, and would be glad if you would tell me how the floors of the large buildings at the British Empire Exhibition were made, as the work I have in hand is very similar.—J. D.

Answer.—The floors of the two buildings at the British Empire Exhibition cover a total of 22 acres, 12 acres in the Palace of Engineering and 10 acres in the Palace of Industry, and are of reinforced concrete 4 in. thick throughout. The site was a sloping one, and as it was a condition that the floors had to be level the higher part of the site is excavated and the material used for filling the lower part. The floors were constructed as follows: After the level had been decided upon, brick piers, 14 in. square, were built on concrete footings on the lower part of the site at 8 ft. 4 in. centres each way and of varying heights so their tops were at the level of the underside of the floor. These brick piers were capped with concrete capitals of the " mushroom " shape used for flat slab construction. The material from the upper part of the site was then excavated and filled in around the piers to form a level area over the whole floor area. Owing to the speed with which the work had to be carried out it was decided to dispense with centering for supporting the floor whilst it was setting, and the concrete was therefore rapidly placed direct on the earth without previously consolidating it. The reinforcement was made up into mats about 9 ft. square, or a little more than the space between each set of four piers spaced at 8 ft. 4 in. centres, and supported on half-bricks placed on the top of each pier, the weight of the reinforcement itself being just sufficient to cause it to sag to near the bottom of the floor in the centre of each mat. It was found that this method of using filled earth as centering answered very well, and only in very few places did the consolidating of the earth result in damaging the slab before it had set. Thus these floors are now resting direct on the ground on the higher parts of the site, whilst on the lower parts they are supported only on the brick piers, the earth having consolidated to several inches below the underside of the floor.

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PRESSURES ON FOUNDATIONS.

By N. L. WALLIS, A.C.G.I., B.Sc.(Lon.), A.M.Inst.C.E.

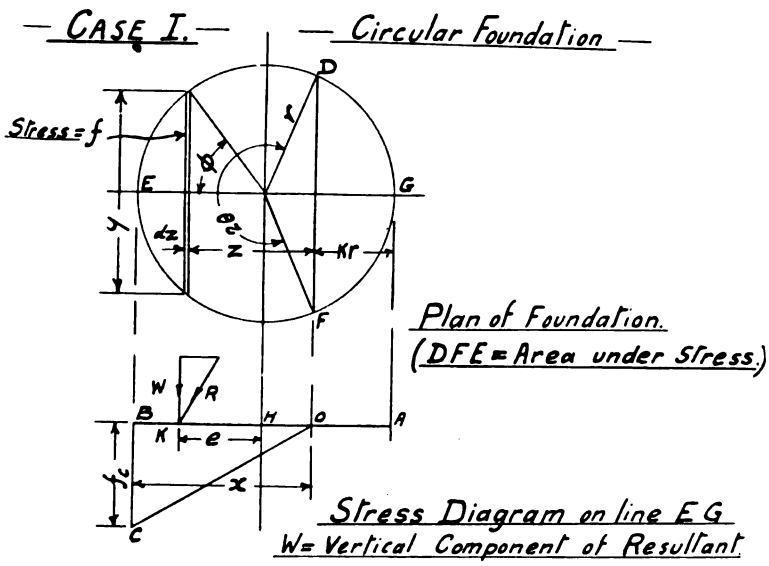
THE following treatise deals with a case of not unusual occurrence, namely, foundations on which the load is so distributed that a portion only of the base is under stress. This obviously assumes that no tension exists. It is customary in text-books to state that the resultant pressure on a foundation must lie within certain limits, but this is only so if it is necessary to maintain a positive pressure over the whole surface. This condition is always necessary in the case of submerged foundations, but on the other hand it is not always necessary in the case of foundations not subjected to water pressure.

In the course of the design of a reinforced concrete tower about 120 ft. high it was found that owing to limited space it was impossible to spread the base sufficiently to maintain a condition of positive pressure over the whole area, and at the same time it was not practicable to provide for the presence of tension on the foundations; these limiting conditions led to the investigation of the stresses in foundations which were only partially under load, and it is thought that the method and the results of this investigation may be of general interest.

The three cases usually met with are:

(1) A circular foundation.
(2) A square foundation with the resultant parallel to one of the sides.
(3) A square foundation with the resultant in the line of one of the diagonals.

Case I.—In the diagram for Case I it is supposed that the resultant "R" strikes the foundations with eccentricity "e," and that "e" has such a value that the stress at G tends to become tensile. But as it is assumed that



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no tension can exist, the area $D\,F\,G$ becomes a region of no stress. $D\,E\,F$ is now the stressed area and $A\,O\,B\,C\,O\,A$ the diagram of stresses on the centre line $E\,G$.

Examination of the stress diagram shows:

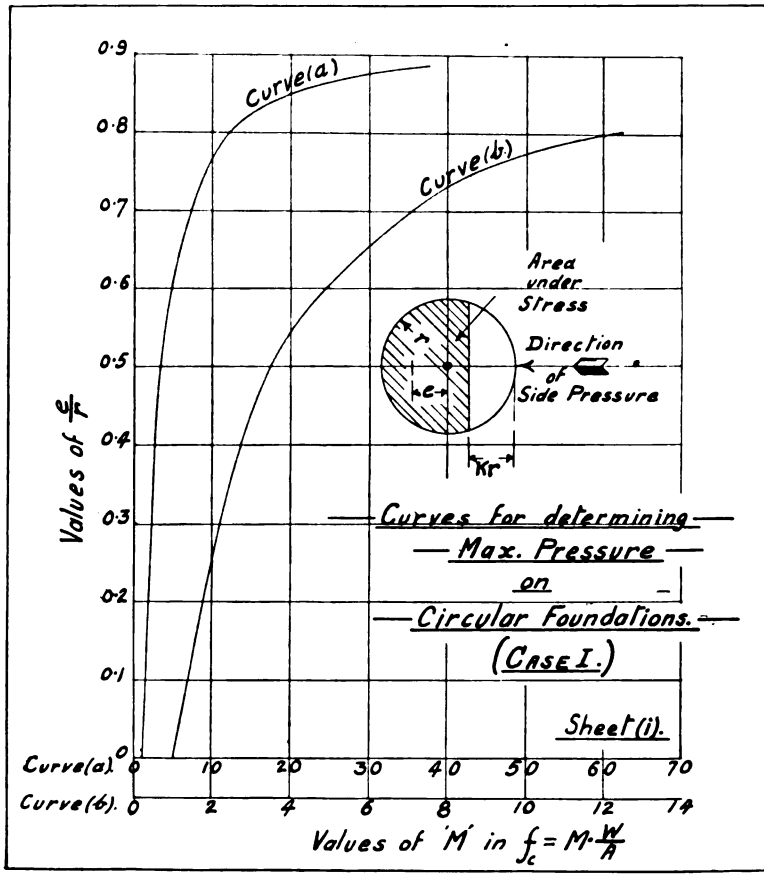
(*a*) That the integration of the stresses in the foundation must equal the total vertical load, i.e., "W."

(*b*) That the moment of this total load "W" about the point O must equal the integration of the moments of the stresses about the same point.

We will deal first with condition (*a*).

We have then:

$$W = \int_{z=0}^{z=x} fydz$$



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And :

$$f = f_c \cdot \frac{z}{x}$$

$$y = 2r \sin \phi.$$

$$z = r \cos \phi + (x - r).$$

$$\therefore f = \frac{f_c}{x}\left\{r \cos \phi + (x - r)\right\}$$

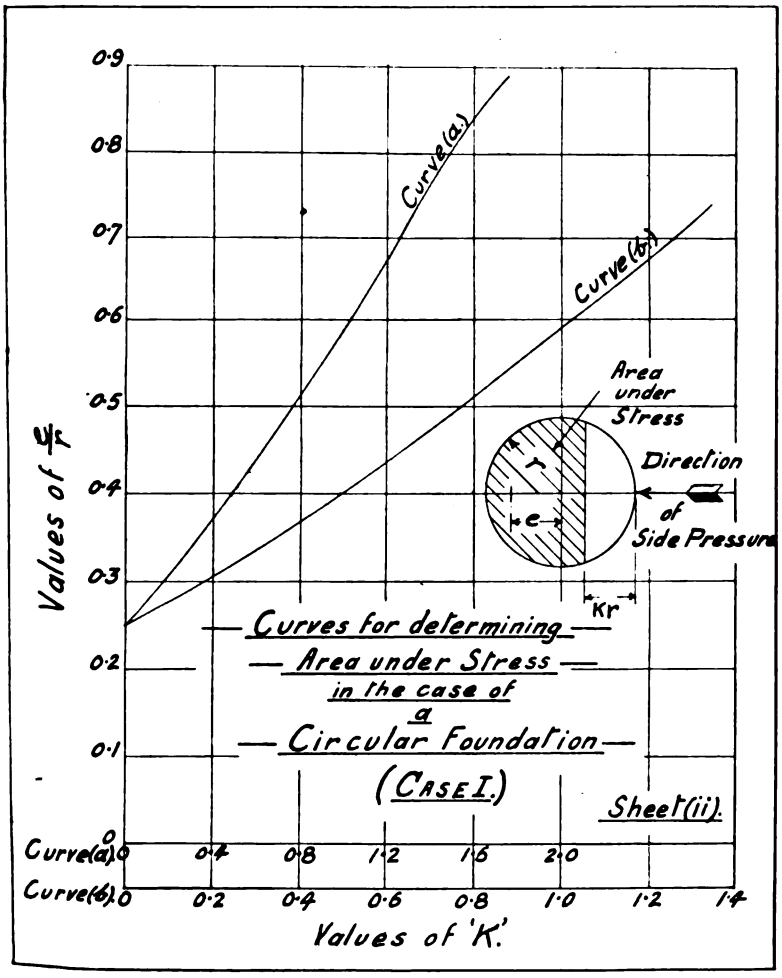
$$dz = -r \sin \phi d\phi.$$

Also when $z = x$ and when $z = 0$

$$\phi = 0 \qquad \phi = \theta$$

$$\therefore W = -\frac{2r^2 f_c}{x}\int_{\theta}^{0}(r \cos \phi + x - r) \sin^2\phi d\phi.$$

$$= \frac{2r^2 f_c}{x}\left[\frac{r \sin^3\phi}{3} + (x - r)\left(\frac{\phi}{2} - \frac{\sin 2\phi}{4}\right)\right]_0^{\theta}$$



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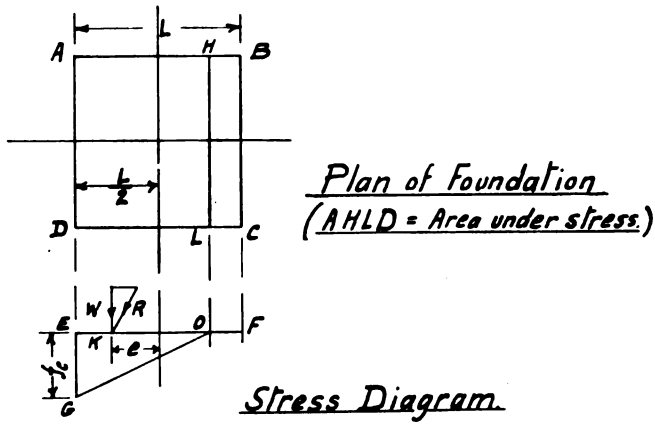
CASE II.

—Square Foundation with Resultant parallel—



Now $x = r(1 - \cos\theta)$.

$$\therefore W = \frac{2r^2 f_c}{1 - \cos\theta}\left(\frac{\sin\theta}{2} - \frac{\sin^3\theta}{6} - \frac{\theta\cos\theta}{2}\right) \quad . \quad . \quad (A)$$

Dealing now with condition (*b*):

Take moments about *O*

$$\text{Then } W(x - r + e) = \int_{z=0}^{z=x} fyzdz$$

and substituting as before we get—

$$W(x - r + e) = -\frac{2r^2 f_c}{x}\int_{\theta}^{0}(r\cos\phi + x - r)^2 \sin^2\phi d\phi$$

$$= \frac{2r^2 f_c}{x}\left[r^2\left(\frac{\phi}{8} + \frac{\sin 2\phi}{16} - \frac{\sin\phi\cos^3\phi}{4}\right) + 2r(x - r)\frac{\sin^3\phi}{3}\right.$$

$$\left. + (x - r)^2\left(\frac{\phi}{2} - \frac{\sin 2\phi}{4}\right)\right]_0^{\theta}$$

Substituting for x and simplifying we get finally:

$$W(e - r\cos\theta) = \frac{2r^3 f_c}{1 - \cos\theta}\left(\frac{5\theta}{8} - \sin^2\theta\cdot\frac{\theta}{2} + \frac{\sin^2\theta}{24}\cdot\sin 2\theta - \frac{5\sin 2\theta}{16}\right) \quad (B)$$

From equations (*A*) and (*B*) the value of f_e can be determined for any given value of e, r, and W.

This can be most conveniently accomplished by means of a curve, as will be shown later.

The conditions (*a*) and (*b*) in Case I are equivalent to the statement that the centre of gravity of the stresses on the base must coincide with the point of application of the resultant.

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In **Case II** this condition is satisfied when :

$$OK = \frac{2}{3}OE.$$

And since "e" is known, we have :

$$KE = \frac{l}{2} - e$$

$$\therefore OE = 3\left(\frac{l}{2} - e\right).$$

And since the integration of the stresses on the base is equal to the vertical component of the resultant we have :

$$W = \frac{3lf_c'}{2}\left(\frac{l}{2} - e\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (C)$$

from which f_c may be obtained for any value of e, l, and W.

Case III.—Square foundation with the resultant in the line of one of the diagonals.

In this case it is necessary to examine the case under two conditions :

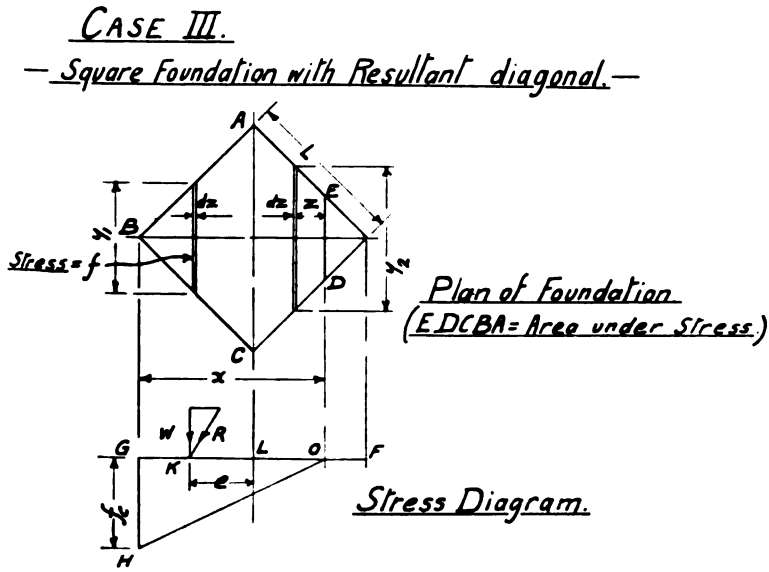
(*a*) When the area under stress is greater than half the base.

(*b*) When the area under stress is less than half the base.

Assuming now condition (*a*).

We have :

$$W = \int_{z = x - \frac{l}{\sqrt{2}}}^{z = x} fy_1 dz + \int_{z = 0}^{z = x - \frac{l}{\sqrt{2}}} fy_2 dz$$



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Now $$y_1 = 2(x - z)$$

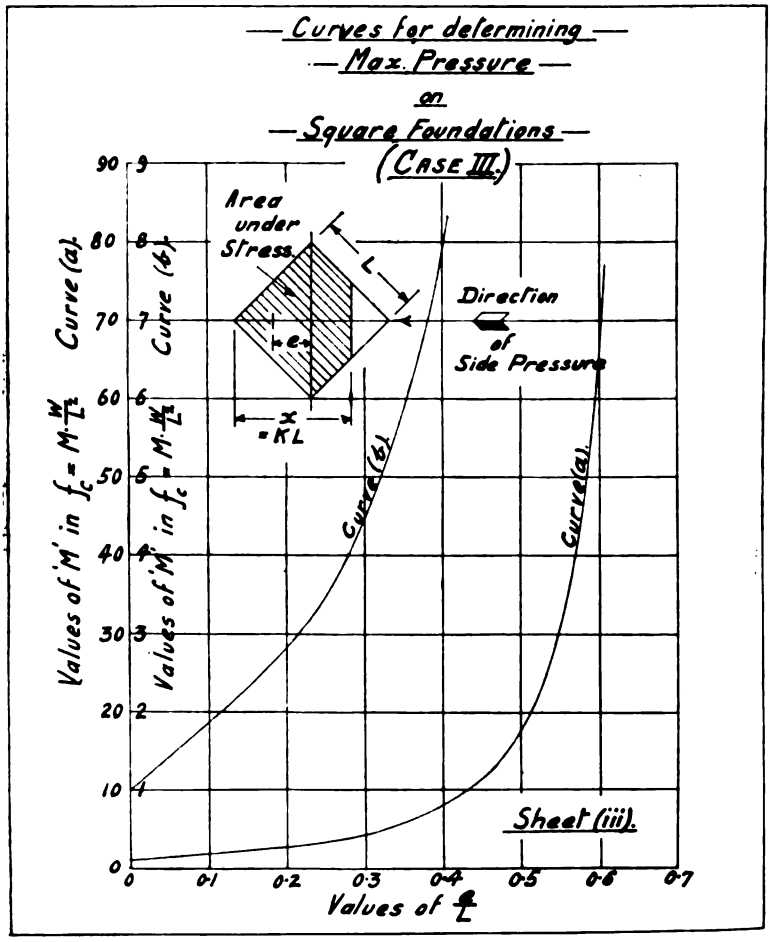
and $$y_2 = 2\{\sqrt{2l} - (x - z)\}$$

also $$f = f_c \cdot \frac{z}{x}$$

$$\therefore W = \int_{x-\frac{l}{\sqrt{2}}}^{x} \frac{f_c}{x} \cdot 2z(x - z)\, dz + \int_{0}^{x-\frac{l}{\sqrt{2}}} \frac{f}{x} \cdot 2z\{\sqrt{2l} - (x - z)\}dz.$$

$$= \frac{2f_c}{x}\left[\frac{xz^2}{2} - \frac{z^3}{3}\right]_{x-\frac{l}{\sqrt{2}}}^{x} + \frac{2f_c}{x}\left[\frac{\sqrt{2}lz^2}{2} - \frac{x2^2}{2} - \frac{z^3}{3}\right]_{0}^{x-\frac{l}{\sqrt{2}}}$$

$$\therefore W = \frac{f_c}{x}\left(-\frac{x^3}{3} + \sqrt{2}lx^2 - l^2x + \frac{\sqrt{2}l^3}{6}\right) \quad . \quad . \quad . \quad . \quad (D)$$



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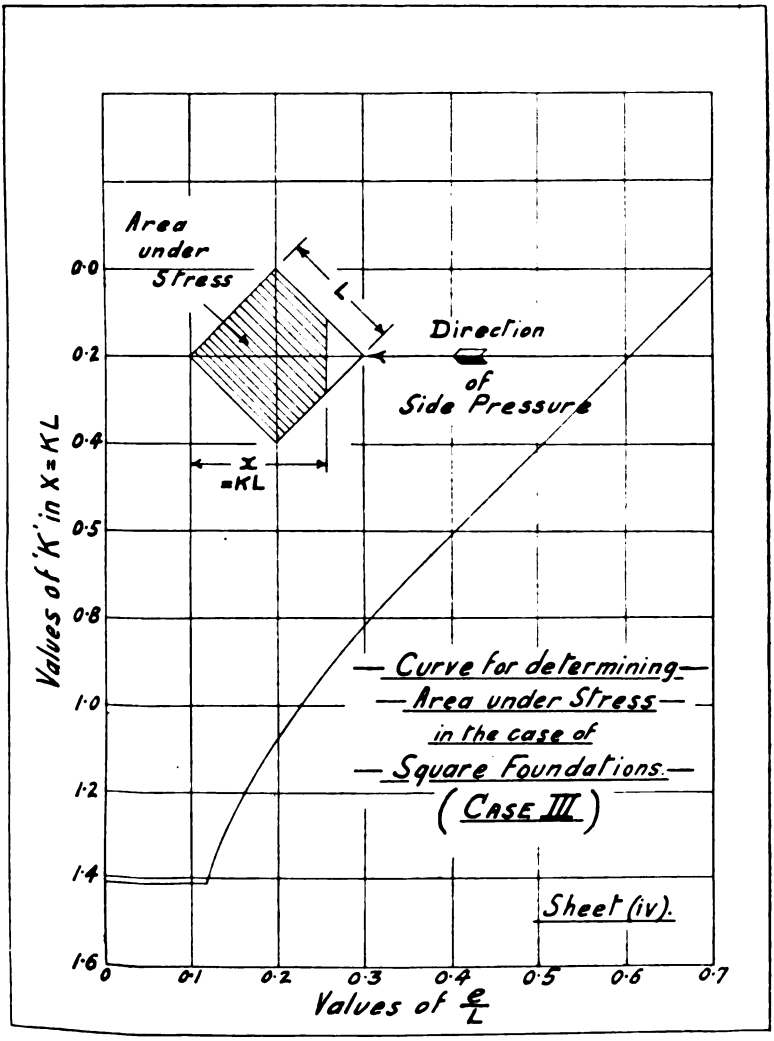
Now taking moments about O we get:

$$W\left(x - \frac{l}{\sqrt{2}} + e\right) = \int_{x-\frac{l}{\sqrt{2}}}^{x} fy_1 z dz + \int_{0}^{x-\frac{l}{\sqrt{2}}} fy_2 z dz$$

which reduces to

$$W\left(x - \frac{l}{\sqrt{2}} + e\right) = \frac{f_e}{x}\left(-\frac{x^4}{6} + \frac{2\sqrt{2}lx^3}{3} - l^2x^2 + \frac{\sqrt{2}l^3x}{3} - \frac{l^4}{12}\right). \quad . \quad (E)$$



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Assuming now condition (b) we have

$$W = \int fy_1 dz\,; \text{ where } y_1 = 2(x - z) \text{ and } f = f_e \frac{z}{x}$$

$$= \frac{2f_e}{x} \int_0^x z(x - z)dz$$

$$\therefore W = f_e.\frac{x^2}{3} \quad \ldots\ldots\ldots\ldots \quad (F)$$

Taking moments about O we get

$$W\left(x - \frac{l}{\sqrt{2}} + e\right) = \int_0^x fy_1 z dz.$$

$$= \frac{2f_e}{x} \int_0^x z^2(x - z)dz$$

$$\therefore W\left(x - \frac{l}{\sqrt{2}} + e\right) = f_e.\frac{x^3}{6} \quad \ldots\ldots \quad (G)$$

Assuming that the value of the eccentricity "e" has already been found, either by means of calculation or, as is generally the case, by graphics, the accompanying curves may be used to determine at a glance:

(1) The value of "m" in the expression $f_e = m.\frac{W}{A}$,

where f_e = maximum stress on foundation
W = total vertical load
A = *total* area of foundation,
i.e. loaded + unloaded areas.

(2) The division of the foundation area into loaded and unloaded portions.

METHOD OF DERIVING CURVES.

Case I.—Take equation (A) and assume any value of θ. Then we get:

$$W = 2r^2 f_e.K_1 \quad \ldots\ldots\ldots\ldots \quad (\alpha)$$

$$\text{or } f_e = \frac{W}{2r^2.K_1} = \frac{\pi}{2K_1} \cdot \frac{W}{\pi r^2}$$

$$= m.\frac{W}{A}$$

where $$m = \frac{\pi}{2K_1} = X.$$

Taking now equation (B) and assuming the same value of θ as before. We get

$$W(e - br) = 2r^3 f_e.K_2 \quad \ldots\ldots\ldots\ldots \quad (\beta)$$

Dividing (β) by (α) we have $e - br = r.K_3$.

$$\therefore \frac{e}{r} = K_3 + b = Y.$$

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"$\frac{e}{r}$" is then plotted as an ordinate with "m" as the abscissa.

Following out this method for all values of θ from 40°–180°, we get the curves shown on sheet (*i*), one curve having 5 times the horizontal scale of the other.

Now for any value of θ we have

$$Kr = r(1 + \cos\theta)$$
$$\therefore K = 1 + \cos\theta.$$

This is plotted against $\frac{e}{r}$ for all values of θ from 40°–180° and the curves shown on sheet (*ii*) are obtained; in this case one of the curves is double the scale of the other to enable the higher values of "k" to be shown.

Case II.—Since this case is, as shown, amenable to direct calculation, no curves are necessary.

Case III.—Take equation (D) and assume $x = kl$, where "k" has any value between O and $\sqrt{2}$.

Then we get

$$W = l^2 f_c K \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\gamma)$$

$$\text{or } f_c = \frac{W}{K_1 l^2}$$

$$= m \cdot \frac{W}{A}$$

$$\text{where } m = \frac{1}{K_1} = Y.$$

Taking now equation (E) and assuming the same value of "x" as before we get

$$W(K_2 l + e) = l^3 f_c K_3 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\delta)$$

Dividing (δ) by (γ) we have

$$K_2 l + e = l K_4$$

$$\therefore \frac{e}{l} = K_4 - K_2 = X.$$

"$\frac{e}{l}$" is then plotted as the abscissa with "m" as the ordinate.

Following out this method the curves shown on sheet (*iii*) are obtained for all values of "k" from 0·2 to $\sqrt{2}$.

One curve has ten times the vertical scale of the other in order to show the higher values of "$\frac{e}{l}$".

For values of "k" from 0·2 to $\frac{\sqrt{2}}{2}$ equations (F) and (G) are of course used in the above calculations.

In order to obtain the curve shown on sheet (*iv*) the value of "k" is plotted against the corresponding value of "$\frac{e}{l}$".

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BOOK REVIEWS.

Reinforced Concrete Bridges. By W. L. Scott, A.M.Inst.C.E.

London: Crosby, Lockwood & Son. Price 42s. net.

OUR views on this work may be summarised by stating that it gives in our opinion a very good treatment of both the theoretical and practical points to be considered in the design and construction of reinforced concrete bridges of all types. The author is chief engineer to the Considère Construction Co., and naturally draws practically all his examples from the bridges which have been designed by this firm, but the treatment is sufficiently general to be of the greatest use to the reader, and we congratulate the author on having prepared a thoroughly useful work on the subject. The use of influence diagrams figures largely in the work in a thoroughly practical manner, and copious examples are given both by photographs and by clear sketches which raise the value of the work considerably. Altogether a book which will be helpful to all those who are specially interested in the subject under discussion.—O.F.

[In the chapter dealing with arch bridges the methods used are considerably simplified from the older methods of calculations, and while the general theory has been treated mathematically the results are in a form that enables them to be applied in practice with a minimum use of higher mathematics. Tables of standard influence line ordinates for the bending moments and horizontal thrust are given. An example of an 80-ft.-span arch bridge fully worked out is given, and attention has been paid to temperature and shrinkage effects in bridges of this class. Several devices are explained and illustrated showing the manner in which arch shortening, shrinkage and similar effects can be accommodated. The standard loading recently adopted by the Ministry of Transport, and applicable to nearly all roadway bridges in this country, is fully described and a method is also given for calculating the roadway slab of a bridge subjected to wheel loading whereby the bending moments in the longitudinal direction of the slab can be ascertained, as well as those resultant from the transverse bending. A chapter has been included on the design of reinforced concrete bow-string girder bridges. A complete specification for a reinforced concrete bridge adaptable to any type is given in the appendix.]

Detailing and Fabricating Structural Steel. By F. W. Dencer.

London: McGraw-Hill Publishing Co., Ltd. Pp. 511; 328 illus. Price 25s. net.

THIS volume has been written with the object of placing before students and others engaged in structural engineering the information gained by the author during his experience; which would, as he states, in the ordinary course take a number of years of association with structural shops to acquire. At the outset the author discusses structural engineering and shop organisations, and describes that of the largest company in the United States, the American Bridge Company, of which concern Mr. Dencer is an engineer.

Under "Bridgework" is dealt the design of details with a view to economy in shop work and material. Then follows a discussion of the various methods of fabricating ordinary materials from the drawings, maximum sizes, and so on. Shop assembly marks and "match-marking," or, as it is called in this country, erection marking, are discussed at length, then follow tables of gauges for various sizes of bars, minimum stagger for driving rivets, field rivet lengths, and bolts.

Further chapters deal in a similar manner with office buildings, mill buildings, machinery for bridges, ships and tank work. The chapters conclude with several typical drawings of actual portions of made-up steelwork, which together with the detail drawings in the text are worthy of careful study. Tank work is dealt with at some length, and there are some useful tables giving rivet spacing and edge distances for different diameters of rivets for oil and water tank construction, also the properties of butt and lap tank joints. The author discusses caulking and connections and field rivetting.

The remaining chapters cover a good deal of ground and relate to export work, material, the progress of material through the shops, the templet shop, punching, reaming, rivetting, hoisting apparatus, inspection, cleaning and rust prevention, forge work, machine shop, shipping, and economical fabrication. These chapters are among the most instructive from a practical point of view and will repay careful reading. The author discusses templet making and the means whereby a

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saving in templets can be made by the use of up-to-date multiple rack and spacing punches, even for repetition work. This chapter is interesting, as showing what is done in America with up-to-date punching machines and tackle.

The inspection of finished steelwork calls for considerable experience, and the author's remarks on shop inspection will be found very helpful to those to whom this duty may fall. In conclusion, the author deals with economical fabrication, and drawing and shop errors. These chapters contain much useful information and discussion on the economical design of details, entirely from a shop or practical point of view. The author has treated all matters dealt with from this standpoint. His book should give considerable help to all interested in structural steelwork, either as students and future structural engineers, as well as to instructors in technical colleges or those more actively engaged in carrying out commercially such work and the volume can, therefore, be thoroughly recommended.—H.F.C.

New Method of Cottage Construction.

Known as the "Newman Mono-block" system, a new method of concrete construction has been patented by Mr. B. Leigh Newman, Lic. R.I.B.A., of "Barnsdall," Barnby Don, Doncaster, and is being used for about three hundred houses now in course of erection in the Doncaster district.

The method is of the pier-and-panel type, variations of which are being successfully used for large numbers of concrete houses in different parts of the country. In the "Mono-block" system the piers are formed of hollow blocks 2 ft. 6 in. high made in three patterns, i.e. for corners, intermediate piers, and for piers at the junction of outer and party walls. These blocks are cast with vertical grooves on two sides for corner and intermediate piers and on three sides for the piers at party walls, and channels are also formed at the top, bottom and sides of each block in which grout is poured to bind the ends of the slabs and blocks together. The piers occur at intervals of three or four feet, or less,

Pair of Subsidy Houses, near Doncaster, finished with dash of Derbyshire Spar.

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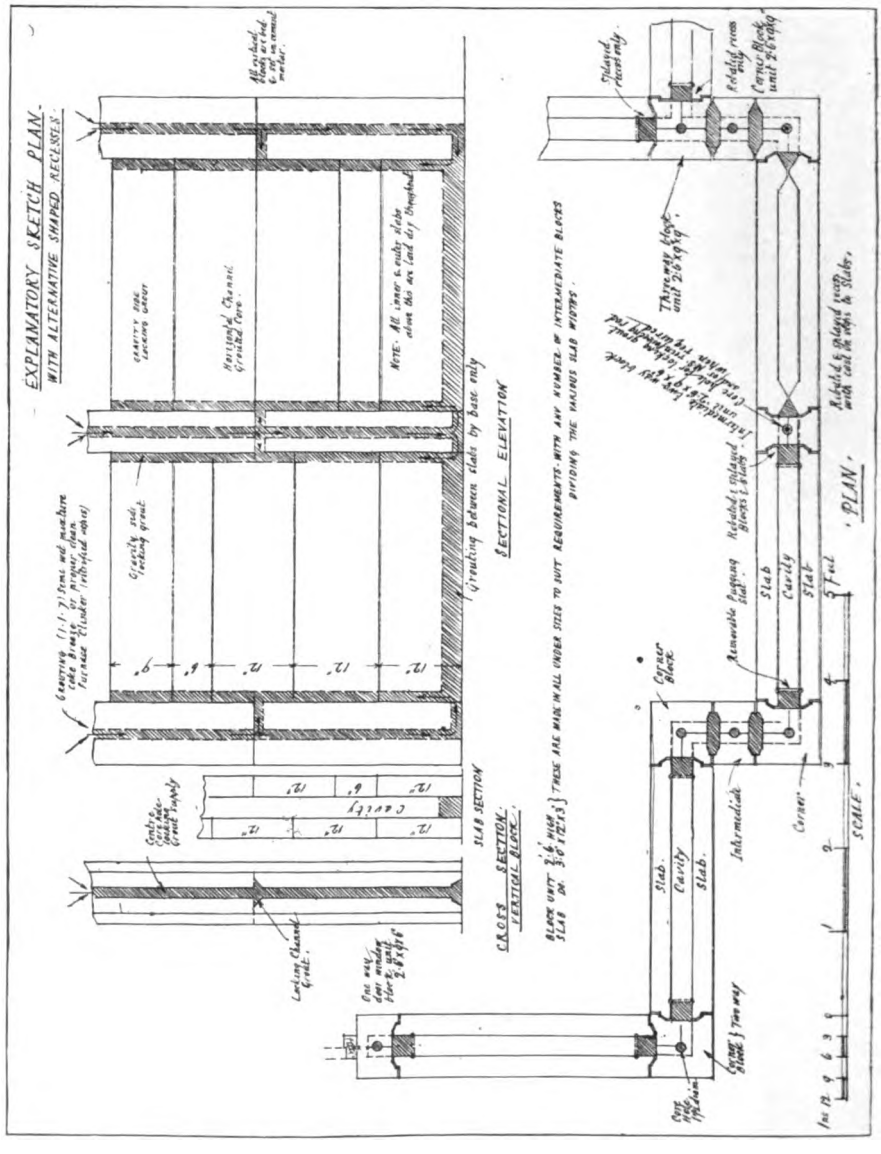
EXPLANATORY SKETCH PLAN.
WITH ALTERNATIVE SHAPED RECESSES.
SECTIONAL ELEVATION
SLAB SECTION
CROSS SECTION.
VERTICAL BLOCK.
PLAN.
SCALE.
Slab.
Cavity
Slab.
Corner Block
Intermediate
Corner
Removable Plugging Slab.
Cavity
Slab
Slab
Cavity
Slab.
Cavity
Slab.
Cavity
Slab.

Part of Block of Four Houses, for the Doncaster Corporation, during Construction.

according to the design of the house, and the intervening spaces are filled in with the slabs to form a double wall with a cavity. The erection of the blocks to form the piers and the slabs proceeds simultaneously, the slabs being laid dry and grouted with cement mortar poured into the channels which receive the slabs. The exterior face is then rendered with stucco, rough cast, or pebble-dash, and the inside with plaster. The whole of the units are made on the site, and under normal conditions the walling of a pair of parlour-type houses can be erected up to the eaves in eight days by this method of construction.

Concrete Floors for Railway Trucks.

According to the *Stone Trades Journal*, experiments are being made in Germany with concrete for floors of railway trucks and cars. We gather that a composition composed of cement and sawdust has been used in the construction of the floors of railway freight cars with considerable success. The first car with this type of floor was built in 1919 in Heidelberg, and tested in the railway freight yards. The car, it is reported, withstood concussion at a speed of 27 kilometres, and after five years of service this test car still remains in perfect condition. The new type of car weighs 20 tons, and in appearance is much the same as an ordinary iron car. The cost of manufactiure is said to be much less than iron, and although the concrete-floored car is heavier the fact that the danger of rust is eliminated offsets this disadvantage, in the opinion of the builders. The car with the concrete floor requires so little repair in comparison with wood and iron cars that the railway administration is favourably impressed. A company for the manufacture of these cars has been formed at Darmstadt, and it is anticipated that production on a large scale may shortly be commenced.

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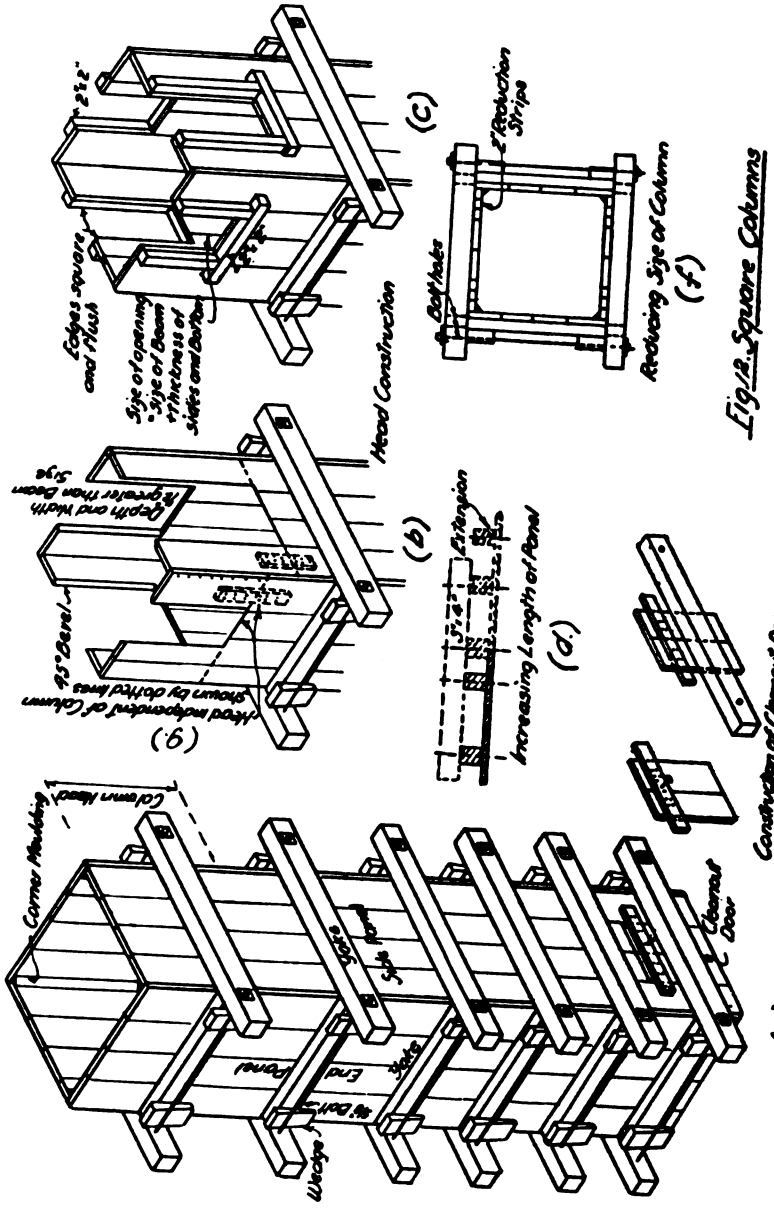

Fig. 12. Square Columns

(See p. 271.)

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DESIGN OF FORMWORK FOR REINFORCED CONCRETE CONSTRUCTION.

By A. E. WYNN, B.Sc., A.M.Am.Soc.C.E.

VII.—DETAIL CONSTRUCTION OF COLUMN FORMS.

There are five usual column shapes, namely, square, rectangular, L-shape, octagonal, and round.

Rectangular columns are nearly always exterior columns and L-shape columns usually exterior corner columns, while the other shapes are used for interior columns.

Except for round columns the general method of framing will be the same for all shapes. The general design of the forms has been shown in previous chapters; the method is the commonest and the most satisfactory and is now considered to be the standard. It is strong, simple to build and strip, easy to reduce, and does not necessarily require any patent device.

The size of timber to use will vary with the height and size of the column and can be chosen from the tables. If a form is only to be used once or twice 1-in. sheathing will be satisfactory, but for use several times 1¼ in. or 1½ in. is better as there will be less breakage and waste. The thicker sheathing, too, will give better lines and surfaces. Since poor lines, fins, bulges, etc., are more noticeable on a column than on any other part of a structure, good timber should be chosen, spruce being very suitable, and generally the thicker the sheathing the better the quality. This is particularly true of exterior columns, perhaps the only exterior concrete that will show, and since they have to be well finished it is economical to frame carefully and choose good timber.

The construction of the column head, or the top part into which the floor frames, will depend upon the kind of floor system used. In a multiple-story building it is usual to build only sufficient forms for all the columns on the first floor and use these over again on the upper floors.

There is no part of the formwork for a building where more money can be saved or lost than in the column forms.

Square Columns (*Fig.* 12).—The method of construction will be the same for rectangular, L-shape, and octagonal columns, with slight variations to suit the particular conditions.

A column form consists of two ends and two sides, each built as a unit or panel, the height of the panel being the story height less the slab thickness less the floor sheathing; this height will include the "head," which may or may not be an integral part of the column form. To determine the spacing of the yokes the height need only be taken to the

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bottom of the deepest beam framing into the column, since concreting should stop there and the concrete allowed to settle before pouring the beams and slab so that there will be no shrinkage cracks between the top of the column and bottom of the beams.

To facilitate making all measurements and to avoid mistakes in measuring, a " measuring stick " is very useful. This stick is cut the full height from floor to floor, and on it successively from the top are marked out the slab thickness, floor sheathing, beam bottom, girder bottom, yoke spacing, and a mark 12 in. up from the bottom. This stick is marked out by the foreman and is used both in assembling the panels and in erection ; it ensures uniformity in measurements, saves the carpenter's time in using a rule, and also prevents his mistakes.

The two ends are built in width equal to the dimension of the column plus twice the thickness of the sheathing, the ends of the yokes and sheathing being flush.

The yokes are first cut to length and laid out on a table at the desired spacing (with the stick) ; then the end board is nailed on at the required distance from the end of the yokes and the remaining boards, except the last one, are clamped tight against the first and nailed ; measurement is then made to determine the width of the last board, which is then sawn and nailed on. To obtain a good finished surface it is essential that the boards be drawn up tight before nailing.

The two side panels are similarly made up, but the width of the sheathing will be the same as the dimension of the column, and the yokes will project 8 in. to 10 in. beyond the sheathing at each end. The panels should be made up an inch or two longer at the bottom to allow for adjustment due to unevenness and variations in floor level.

The spacing of the yokes must be alike on both ends and sides. Bolt holes are bored in the side yokes about $\frac{1}{8}$ in. larger than the size of bolt, and the spacing is calculated from the size of column and timbers used, allowing about $1\frac{1}{2}$ in. for the wedges. If there is a boring attachment on the machine saw the holes should be bored before the sheathing is attached to the yokes. On a large job each operation should be the work of one man or group of men—sawing and boring the yokes, sawing the sheathing, nailing to the yokes—thereby saving much waste labour, all carrying of timber to the carpenters being done by labourers.

The panels are stacked up ready to be carried into place as required, and they should be oiled at this time so they will not swell and warp if left long in the rain and sun before being used.

Column Heads.—In mushroom slab construction there are no beams and the column will finish off with a splay cap independent of the column, so that the height of the column will be measured to the

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bottom of the cap and the head of the column will be cut off square, all panels being of the same length (*Fig.* 12 (*a*).)

In other types of construction there will be beams framing into the head, and the head can either be formed as a separate unit or as part of the column form. In the former method (*Fig.* 12 (*g*)) all the panels are stopped off at the bottom of the beams framing into them—these may be of different heights—and framing the head of the column from the bottom of the beams to the underside of the slab separately from the column. The length of each panel will be measured to the underside of the beam, allowing or not for the thickness of the beam bottom, as will be seen later.

There are several advantages in this method, such as ease of stripping since the column head is not part of the column form ; reduction of panel width on upper floors need only be made on one side of the panel instead of both ; and there is no risk of the openings for the beams being cut to the wrong size, since the head will be formed after the beam bottom is in place.

The head is formed by building four corners of sheathing, nailing each corner together, and cleating them on to the column sheathing. They will be held on the sides by the beam sides, and at the top by the floor sheathing, and as an additional brace a 1 in. by 6 in. is nailed diagonally across the corner from beam side to beam side. In stripping, the cleats are knocked off and the head remains in place while the panels are stripped.

If the column reduces on upper floors half the reduction is sawn off each leg of the corner.

When the head is an integral part of the column form the openings for the beams are cut in the panels at the time they are being made up. The opening for a particular size beam should never be the exact dimensions of the beam but always greater, otherwise when the beam sides give slightly under the pressure of the concrete it will be found that the edges of the openings in the panels will be embedded in the concrete and it will be impossible to strip them. For the same reason the beam sides should not be brought flush with the inside of the column sheathing, as in this case the ends of the beam sides will be embedded in the concrete.

One method (*Fig.* 12 (*b*)) is to make the opening $\frac{1}{4}$ in. to $\frac{3}{8}$ in. larger all around than the size of the beam, and bevel the side and bottom edges inwards at about 45 degrees. The beam bottom and sides will butt up against the column sheathing, and the $\frac{1}{4}$-in. clearance will allow for the "give" in the forms.

Another method (*Fig.* 12 (*c*)) is to increase the opening by the thickness of the beam sides and bottom and to nail 2 in. by 2 in. cleats all round flush with the opening.

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The main thoughts to bear in mind when building the column head are ease of stripping and allowance for "give" in the forms.

Erection.—If the forms are not too high and heavy panels can be assembled and bolted up near where they are to be erected and then lifted into place as a unit, this is better than erecting each panel separately and will save some temporary bracing. The column reinforcing is built up into a unit on the floor above and dropped into the column form. If, however, the forms are heavy or have to be lifted high over the reinforcing steel from the column below, then each panel must be erected separately.

Since there is often an inch or so variation in the level of the concrete at the different columns, especially when the finished floor is put on afterwards, a mark should be given with the instrument on the projecting column or footing steel at, say, 12 in. above the finished floor line, otherwise unevenness or a mistake at one point may be carried up throughout the building. This mark is compared with the mark on the measuring stick, which is also 12 in. up from the floor, and gives the amount to saw off the bottom of the column form before erecting it.

When the form is in place and the bolts tightened, two wedges, preferably hardwood, cut out of 2 in. by 4 in. about 6 in. long, are driven in on each end between the bolts and yokes to bring the ends up tight against the side sheathing.

The use of bolts and washers for tying the panels together is considered standard practice, although there are some excellent patent clamps on the market, which will be mentioned later. Bolts and washers are cheap and can be purchased anywhere, and they are more adaptable to different conditions than other forms of ties and clamps. The standard size is $\frac{5}{8}$ in. diameter bolt with 3 in. square by $\frac{1}{8}$ in. thick wrought-iron washer.

Alignment and Bracing.—It is usual to set exterior columns first accurately and locate the others from these, working towards the centre of the building. After the exterior column forms are set the girder bottoms are nailed on from these to the first row of interior columns, not driving home the nails. This is done in both directions and serves to brace the forms and set them very closely to their correct position. To get the exact positions the measurements are made with a steel tape along the girder bottoms, starting from the building line. This will fix the top of the columns exactly, and they must then be plumbed by suspending a plumb-bob, say, 6 in. out from the sheathing and measuring from this to the sheathing at the bottom. If for any reason the interior forms must be set first they must be temporarily braced; this can be done by the method described for rectangular columns.

Changing Height of Forms.—When re-using a form it may be neces-

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sary to increase its height. This should be done at the bottom, because the pressure there will be greater, requiring a closer spacing of yokes, and also to avoid changing the head if it is part of the column form.

The first extension yoke is put on the bottom of the panels so that the end of the sheathing comes halfway on the yoke and the joint in the sheathing at the centre of the yoke. To increase the stiffness at the joint vertical 3 in. by 4 in.'s long enough to cover four yokes can be bolted symmetrically across the splice, two to each side, using the yoke bolts (*Fig.* 12 (*d*)).

To decrease the height it is only necessary to saw off a length at the bottom of the form, but if this leaves the lowest yoke too high up an additional yoke must be added about 4 in. up from the bottom.

Re-erection and Reducing Forms.—When using forms again on upper floors it is generally necessary to reduce their size. The column number should be marked on the panels as a means of identification. After the forms are stripped the panels are carried to the building line and raised to the floor above and placed in their proper location.

If the head is part of the column form the reduction of each panel must be made symmetrically about the centre line, otherwise the beams will be off centre. The two ends are reduced by sawing off a strip from each end of the panel equal to half the total reduction, sawing through both sheathing and yokes. The two side panels are reduced by taking off the two outside boards, one on each end, and replacing them by narrower boards. It is good practice to allow for successive reductions at the time of making up the side panels by having the end boards of widths equal to the successive reductions; it is then only necessary to take off the two outside boards at each reduction (*Fig.* 12 (*f*)).

New bolt holes will be necessary, spaced from the old ones a distance each end equal to half the reduction. It is best to bore all the holes that will be required when the panels are made up. If there are only two stories and one reduction to make, two 2 in. by 4 in., or whatever the reduction requires, can be placed vertically against the end panels between the bolts and yokes and the bolts wedged tight against them.

If the head is independent of the column, or if there are no beams framing into the column, the whole reduction can be made on one end of the panels.

Corner Moulds.—Sharp corners are easily broken when stripping, so it is always advisable to use a triangular bevel strip in the corners; these should be nailed to the sides, not the ends (*Fig.* 12 (*a*)). A rounded corner mould looks well but is much more expensive; if used it should be a part of the end panel sheathing, the wedges bringing it tight against the side sheathing.

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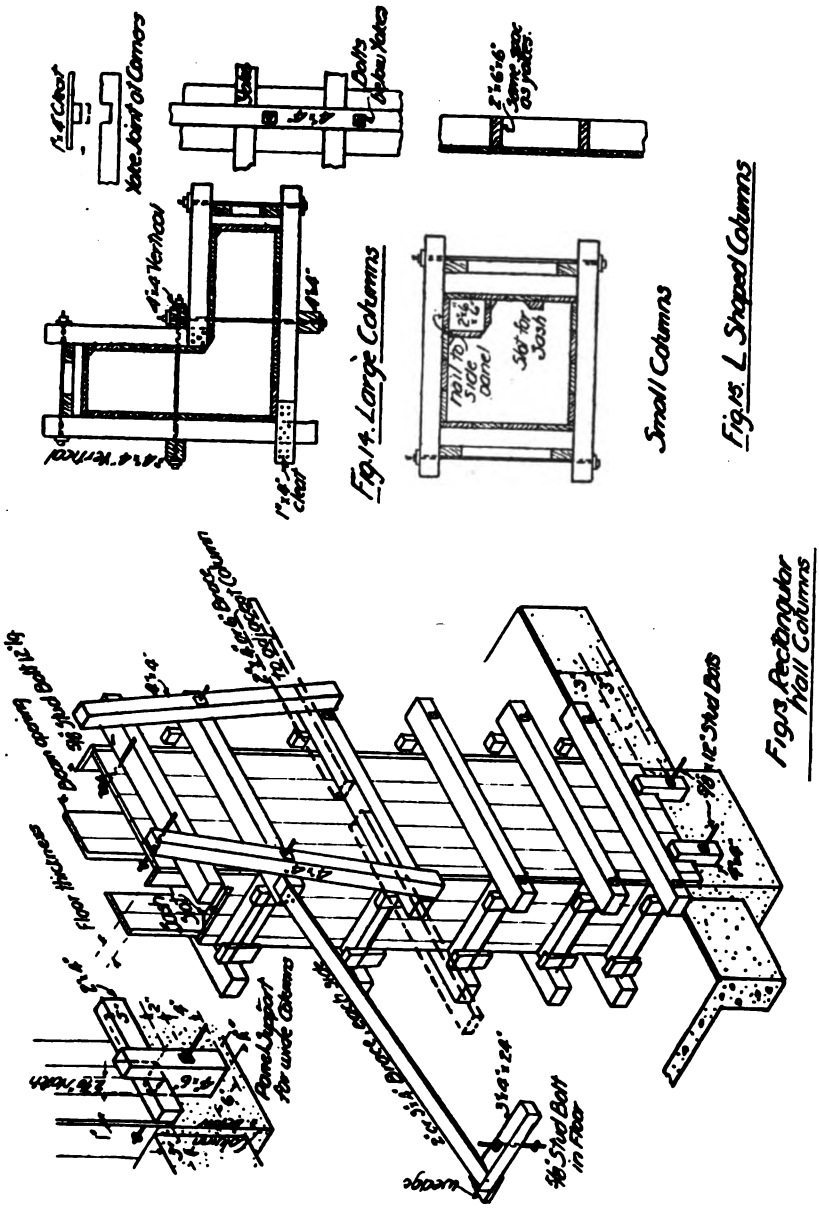

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Clean-out Holes.—A column form is a convenient receptacle for all the shavings and refuse on the floor, so some provision must be made in all forms for cleaning out the form through a hole at the bottom. This detail must be decided upon and built into the panels when they are made up. The hole should be about 12 in. square, depending on the size of the column.

There are two common methods of framing this detail. In the first method the hole is cut out of the sheathing and the loose piece nailed on again with battens, leaving the nails projecting. The tongues are cut off so that the loose piece can be slipped back easily, and a long nail should be driven into the batten to form a handle. When ready to clean out the column the loose piece is slipped off and nailed on to the form to prevent it being lost, it is easily put back again.

The other method is to cut out the hole in the sheathing and nail the lowest yoke to the loose piece only, and not to the adjacent boards. A batten across the loose piece will hold it in place until it is required to move it, when the loose piece and yoke are taken off (*Fig.* 12 (*e*)).

Cleaning should be done by a steam or water hose from above.

Rectangular Column Forms (*Fig.* 13).—These are usually exterior columns, the long side being parallel to the building line and remaining the same throughout all floors, while the short sides or ends often reduce from floor to floor. In general they are built up in panels the same way as square columns, but there will be a few differences since the outside panel will overhang the column below, and the whole form must be carefully braced to prevent tipping over.

The outside panel should be 3 in. longer at the bottom than the inside panel so that it will lap over the column below, and at the top it must reach to the top of the floor slab. The inside panel will be the same as for an interior column. The two end panels will have the wall beams framing into them, and the sheathing on the outside of the beam must reach to the top of the slab and on the inside to underneath the slab sheathing, unless the head is independent.

For the first story, if the column starts from a footing, the erection will be the same as for an interior column, but if it starts flush with a foundation wall, and for all upper stories, special means must be used to hold the outside panel in place. If it starts from a footing the column will generally be longer than required for the next floor, so when reducing the outside panel is left 3 in. longer than the others; if it starts from a wall this panel is originally made up 3 in. longer.

The outside panel is first erected, and since it will project beyond the edge of the slab it must be supported at the bottom until the other panels are erected and bolted up. This support is provided by embedding

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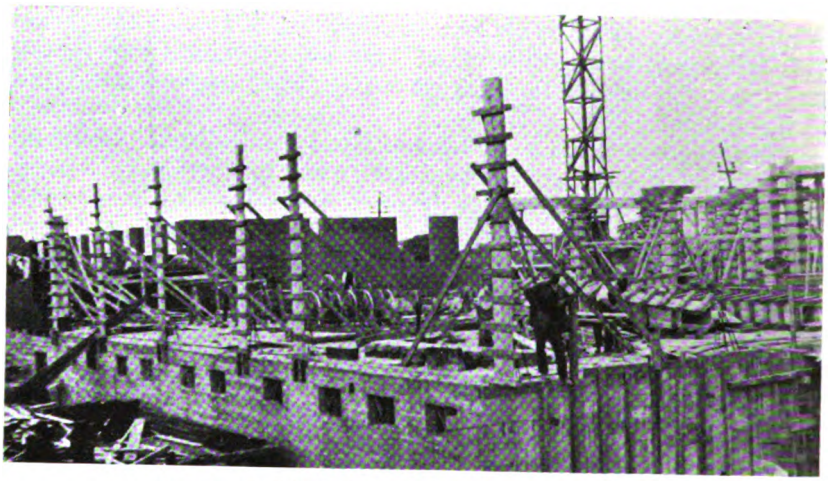

FIG. 16.—EXTERIOR PANELS PLUMBED AND BRACED.

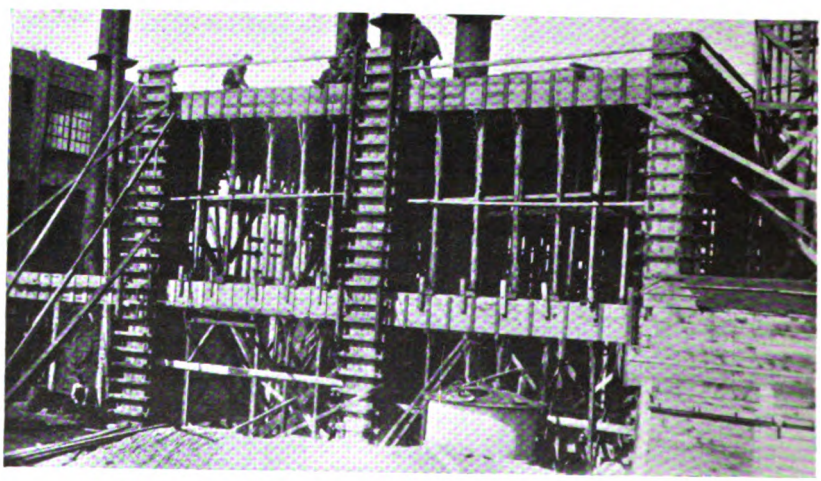

FIG. 17.—COLUMNS 30 FT. HIGH: VERTICAL OUTSIDE BRACES.

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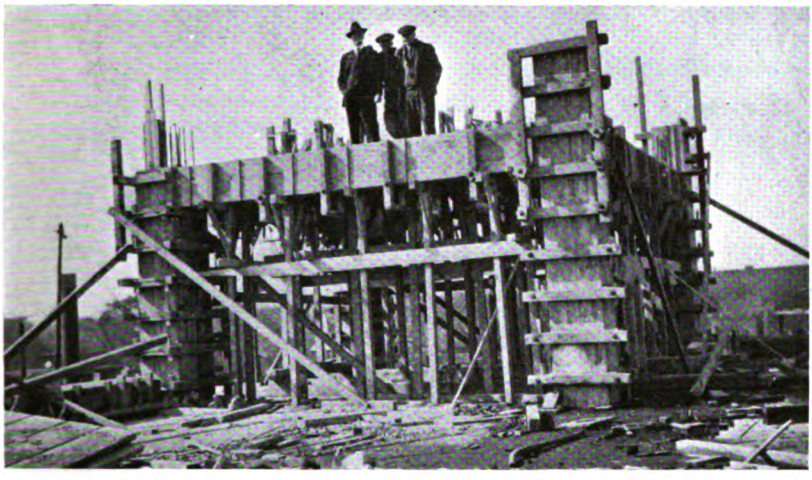

FIG. 18.—BRACING AT TOP OF UNSUPPORTED PANEL AND DIAGONAL BRACING TO BLOCKS BOLTED TO FLOOR.

two $\frac{5}{8}$ in. stud bolts 12 in. long, threaded each end in the concrete, about 6 in. below the top of the wall or column below at the time it is poured. The bolts will project from the concrete 6 in. to 8 in. (with a nut on the end in the concrete) and must be well greased or oiled. The distance apart of the bolts will be about 12 in. less than the width of the column. Over each bolt is slipped a piece of 4 in. by 4 in. about 8 in. to 12 in. long, with a notch cut down one side so that the bottom of the notch will be 3 in. below the top of the column. The depth of the notch should be the thickness of the column sheathing less $\frac{1}{8}$ in. The outside panel is slipped into these notches, and when the 4 in. by 4 in.'s are bolted up the bottom will be securely held and the panel drawn tight against the concrete. With very wide panels, say, 3 ft. or more, to be sure that the whole panel is drawn tight against the concrete a 2 in. by 4 in. is placed between the notch and sheathing; the depth of the notch will then be $\frac{1}{8}$ in. less than the combined thickness of the 2 in. by 4 in. and sheathing.

Before the panel is erected a 2 in. or 3 in. by 4 in. is nailed on to the end of a yoke near the top, one on each end, and the panel is held by these braces while being put in place. When the panel is plumbed the other ends of these braces are nailed to wooden blocks, fastened to the floor, either to the top or side of the block. Provision should always be made when pouring the floor to provide means for holding these blocks. It is best done by leaving stud bolts at the required places

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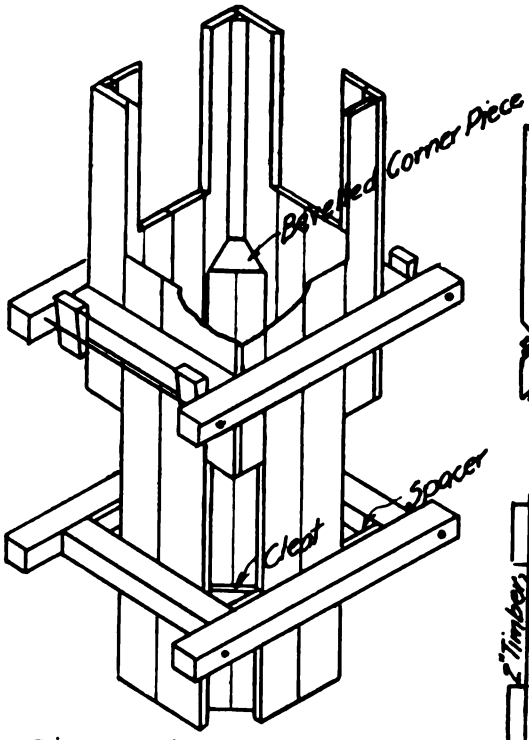

Fig. 19. Octagonal with Square Head

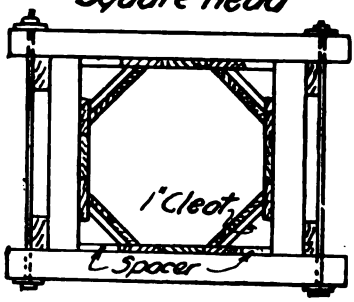

Fig 20. Octagonal Column

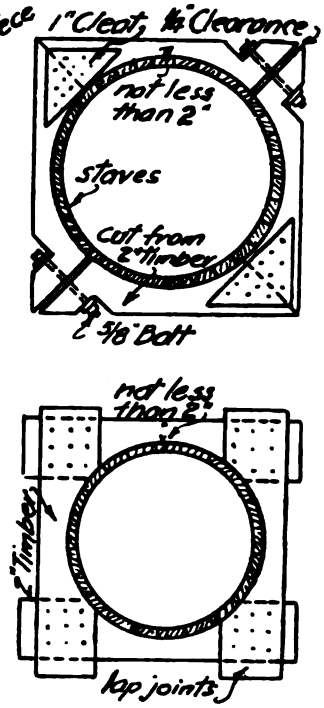

Fig 21 Round Column

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in the concrete, with nut down, and well greased and projecting 6 in. above the floor. Over these bolts short pieces of 3 in. or 4 in. by 4 in. can be slipped and the nuts screwed tight; a wedge driven in between the timber and the floor will prevent it turning.

Another method is to twist some wires around the reinforcing steel and leave the ends projecting to be twisted around the 4 in. by 4 in. Any stud bolts left in the concrete, either in floors or columns, if well greased, are easily twisted out when finished with and the holes filled with mortar so that they will not show.

When the outside panel is plumbed and braced the two end panels and the interior side panel are erected and all bolted up. Two-inch by 4 in. or 6 in. braces, depending on the depth of yoke, should extend from column to column along the outside to keep the forms in line in that direction.

The outside panel of the column head must next be braced, since there is no beam entering it. The top pair of yokes that can be bolted together will be below the deepest beam framing into the head, and since there will be one outside yoke above these it must be supported by other means. This is done by bolting to the top pair of bolts two vertical pieces of 4 in. by 4 in. long enough to bear on the three top yokes. Instead of using the yoke bolts two short bolts can be used, one on each end, making this yoke about a foot longer to allow for them; by this means when stripping the form the two vertical pieces will always be attached to the panel and so will not become lost. This same method is used for interior columns when one panel of the head is unsupported, and can be used for end panels by placing the vertical pieces between the wedges and the yokes.

It is usually necessary to attach steel sashes to exterior columns, and to do this a slot must be left in the side of the column into which the sash is slipped and grouted. These slots are made by nailing a 2 in. by 4 in., bevelled at 45 degrees on the inside edge, to the end panels for the full height of the sash. It may be necessary to bond a partition wall into the column, in which case a similar 2 in. by 4 in., but bevelled on each edge, is nailed to the column form on the centre line of the partition.

L-shape Columns (*Figs.* 14 and 15).—These are almost always corner columns. If the difference in width between the short and long sides is not over 12 in. it is best to build up a square column form with sides equal to the long side and then block out for the "L" in one corner. The corner piece is made by nailing sheathing on to 2 in. timber spaced the same distances apart as the yokes. It is built as a unit and then nailed to one side panel.

Corner columns are often very large for architectural effect, and

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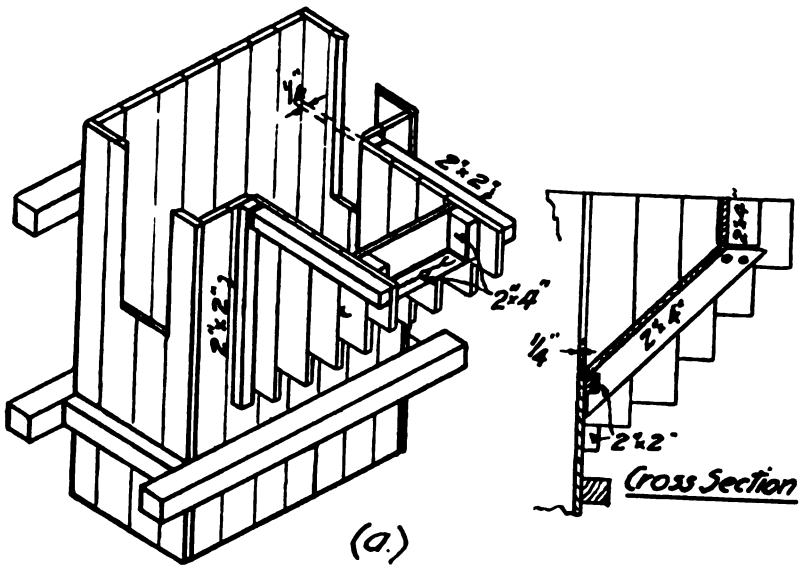

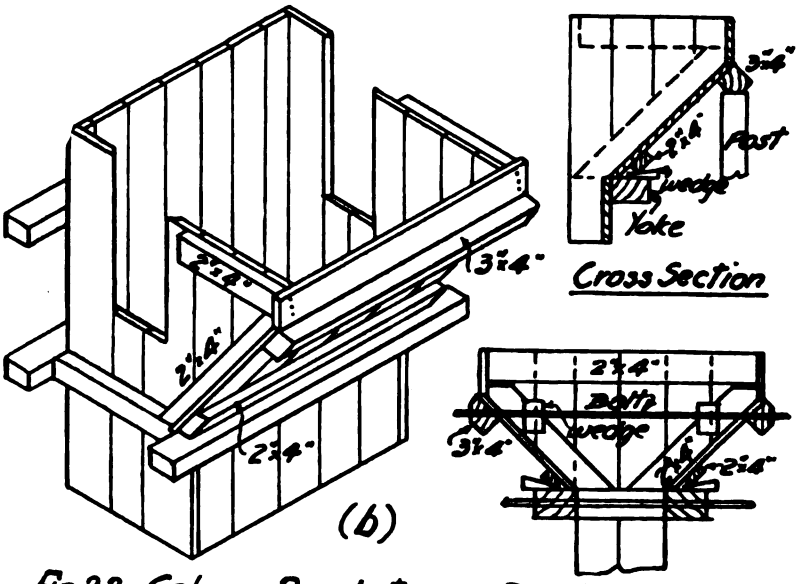

Fig. 22. Column Brackets

Double Bracket

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another method must then be used. There will be six panels to a complete form, the yokes being spaced similarly on each. The yokes on the two long and two short sides are notched over each other at the corners, those on the long sides overlapping, with a short piece of 1 in. by 6 in. nailed over the joint. The two ends are built and bolted in the ordinary way. As the sides will be long, one or two intermediate bolts will be necessary. These bolts should pass through vertical 4 in. by 4 in. or 6 in.'s outside the yokes to stiffen the form, one of them being at the inside corner. The panels with the notches cut in the top of the yokes must, of course, be erected first. The panels are supported and braced in the same manner as rectangular columns, only on two faces instead of one.

Octagonal Columns (*Fig.* 20).—An octagonal form is a modification of a form for a square column.

Four panels are made up as for a square column, but the sheathing need only be 4 in. wider than the side of the octagon, except that at each side yoke the sheathing is blocked out with spacers to make the total width equal the diameter of the column plus twice the thickness of the sheathing in order to give bearing for the end yokes. The remaining four sides are single boards of the required width or two or more boards cleated together, with the sides bevelled at 45 degrees. The loose sides are then nailed in place at the yokes just sufficiently to keep them in place. The side of an octagon is 0·414 times the diameter.

To reduce the form it is necessary to saw a strip off one edge of the loose sides and both ends of the end panel yokes and to reduce the width of the spacers.

The head of an octagonal column is often square to receive the beams. In this case the head is fully sheathed as for a square column and the four corners are filled in with pieces of sheathing, or, if small, with solid pieces cut to fit (*Fig.* 19).

Round Columns (*Fig.* 21).—Wooden forms for round columns are seldom used unless there are only a few columns, since steel forms, which can often be rented, are cheaper.

In order to be able to strip easily and to use the forms over again the following method can be used. The yokes are cut out of 2 in. timber, the width of the boards depending on the diameter of the column; a minimum of 2 in. should be left after cutting out the circle. The yokes are cut as in *Fig.* 21, the diameter being the diameter of the column plus twice the thickness of the sheathing. The joints in the yokes are made on two diameters at right angles; two opposite joints are nailed together with 1 in. cleats and the pair at right angles are bolted together, notches being cut in the yokes so that the bolts will have square bearings.

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There should be at least ¼ in. clearance at the bolted joints so that the joints can be drawn tight.

The sheathing will have to be stave-shaped to the desired radius unless the column is very large, when 1 in. by 4 in.'s can be bent around the circle and any sharp edges afterwards rubbed off. The spacing of the yokes will depend on the safe span of the sheathing, which can be obtained from the tables.

Instead of the two butt joints and battens, a joint with less strength, but sufficiently strong for small columns, is made by overlapping the yokes and securely nailing them together. If the form is only to be used once the bolts can be dispensed with and all joints made butt or overlapping. These forms cannot be reduced without making new yokes; new staves will not be necessary unless the reduction is very large.

Very High Columns.—Columns of more than the ordinary heights up to 12 ft. should be braced or stiffened at intermediate points to give them sufficient rigidity. Long outside columns should have vertical 3 in. by 4 in.'s on the outside yokes, the bolts being long enough to pass through them. Inside columns can be stiffened in the same way or by diagonal braces.

Stripping Forms.—Columns are stripped from two to three days after pouring. The concrete should be sufficiently hard so that the corners will not be broken off in stripping, but should not be so hard that it cannot easily be rubbed smooth. The common test is to strip when the finger nail will not dent the concrete, or when the concrete rings to a blow of a hammer.

To strip, remove the wedges first, then the bolts, and pry out the bottom of the side panels until they are free of the beams, when they can be let down. Stripping should not start at the top of the column but at the bottom. Wherever possible pry on timber and not on concrete.

Brackets (*Fig.* 22).—Brackets are often built as an integral part of the column form, as in mushroom slab construction, and when brackets are required to carry a crane girder. Two methods of construction are shown in *Fig.* 12; (*a*) is more suitable for small brackets and (*b*) for larger brackets the width of the column.

In (*a*) the bottom is carried on 2 in. by 4 in.'s nailed to the sides, which are nailed to the cleats around the opening. The sides must be held by a bolt or wire to prevent spreading.

In (*b*) the sides are cut to the slope, with 2 in. by 4 in.'s nailed along the top and slope. The bottom is a separate panel of sheathing nailed to a 2 in. by 4 in. at the bottom, which is wedged up from the yoke against the sides, and to a 3 in. by 4 in. at the top, which is supported by a post

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at each end or bolted through to the column yoke at the back. The posts may be carried on a yoke, in which case they should be braced from the side forms. If there is a double bracket the 3 in. by 4 in.'s at the top are bolted together. Note that there should be ¼ in. clearance between the bracket and column sheathing.

Cost of Column Forms.—Column forms require 0·2 to 0·3 cubic feet of timber per square foot of contact, and an average value can be used for estimating. The larger the column the less the timber required per sq. ft. It is usual to allow sufficient timber to form all the columns on the lowest floor and to use the forms again on the upper floors. The timber required will therefore be the surface area of the lowest story columns multiplied by the above unit. If an upper story is higher than the lowest, this quantity should be multiplied by the ratio of the heights to allow for splicing out.

The forms should be good for use at least six times, after which some allowance should be made for renewals, say, 15 per cent. per floor.

Making up panels and erection will require about 1 labourer to 2 carpenters, and stripping will require 2 labourers to 1 carpenter. The stripping may be done by labourers only, but a carpenter will take more care of the forms and it will pay to employ one.

To complete 100 sq. ft. of interior square column forms will require 4 hours carpenters' time and 2 hours labourers' time making the panels; 8 hours carpenters' time and 4 hours labourers' time for erection, aligning, and plumbing; and 1½ hours carpenters' time and 3 hours labourers' time for stripping. Cleaning forms and hoisting to next floor will each require about 1 hour labourers' time.

Cost of 100 sq. ft. of forms, 1st erection—
13½ hours carpenter at .....
9 ,, labourer at .....
Timber, 100 × ·25 = 25 cu. ft. at .....

Cost of 100 sq. ft. of forms, each floor above—
9½ hours carpenter at .....
9 ,, labourer at .....
No timber will be required.

Exterior columns will require about 25 per cent. more time to erect.

Octagonal columns will cost about 50 per cent. more than square, and round columns about 100 per cent. more per sq. ft.

Brackets will cost about 50 per cent. more per sq. ft. than square columns.

(To be continued.)

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The Foreman's Guide to Concrete.—VII.

A MANUAL FOR ALL ENGAGED IN CARRYING OUT CONCRETE AND REINFORCED CONCRETE STRUCTURES.

By Albert Lakeman, M.S.A., M.I.Struct.E.

LEVERAGE OF BEAMS.

THE principles of leverage have been explained, and the application of the principles to ordinary beams can now be shown in relation to the reactions at the supports and the bending moment at different points in a beam.

The beam is not, of course, required to move or lift the weight as is the case with the crowbar used to illustrate the previous examples, but it is necessary to balance the weight. The pressure that has to be dealt with in the case of a reaction is not an actual upward force but a resistance, on the part of the materials forming the support, which is exerted when the weight is applied at the ends of the beam. The pressure on the support which causes the reaction is due to the beam acting as a lever, and this can be illustrated in two different ways.

In the first instance we can assume a simple beam which carries a central concentrated load, as shown in *Fig.* 46. In

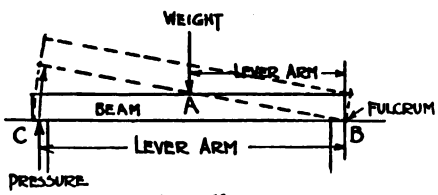

FIG 46.

this case the beam can be considered as a lever having a total length equal to the span which is the distance from C to B, and the weight is situated on the lever at the point A. This is a lever of the second order where the weight is between the pressure and the fulcrum, because an upward pressure at the support at C would tend to lift the lever, as shown by dotted lines, and the fulcrum would be at the other support as shown at B. This means that the lever arm of the weight is equal to the distance from A to B, which is one-half the span, and the leverage of the pressure at the support is equal to the distance from C to B, which is the whole span. The relation between the weight and the pressure will depend on the ratio of the lever arms or the position of the weight or load on the beam. If, for example, the span of the beam is 10 ft. and the load is placed at the centre of the span, then the lever arm of the load will be 5 ft. The pressure at C is therefore equal to the weight A multiplied by 5 ft. and divided by 10 ft., and if the load is 10 tons the pressure is then equal to

$$\frac{10 \text{ tons} \times 5 \text{ ft.}}{10 \text{ ft.}} \text{ which equals 5 tons.}$$

The pressure at C is therefore 5 tons, and this is the reaction at the support; or in other words the amount of resistance that must be put forward by the strength of the material at the support. If the position is reversed so that the fulcrum is considered at the other end, and the pressure at the right-hand support is calculated, it will be clear that the lever arms are the same length as before and the pressure at the right-hand support is also 5 tons. This shows that as the load is central on the beam one-half of the load travels in each direction to cause pressure on the supports, and the total pressure on these is equal to the total load. In all these explanations the weight of the beam itself is neglected in order to avoid complications.

In order to show a further application of the leverage principle the same beam can be considered as shown in *Fig.* 47,

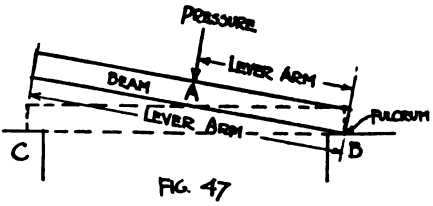

FIG. 47

where the load or pressure is given as acting downward with the fulcrum at B as before to cause a weight or pressure on the support at C. If considered in this way the example is a lever of the third order as the beam is acting in a

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similar manner to the treadle of a sewing machine (*Fig.* 45).

Taking the load or pressure at 10 tons as before, placed at the centre of a beam 10 ft. long, the weight or pressure caused at the abutment C is 5 tons, as before calculated, because the ratio of the lever arms is 1 to 2 and the load loses in value as regards the pressure on one support in the same proportion as the ratio of the lever arms. In calculating the reaction or pressure on a support the position of the load affects the amount because the nearer the load the greater the reaction. As a simple example we can consider the beam shown in *Fig.* 48,

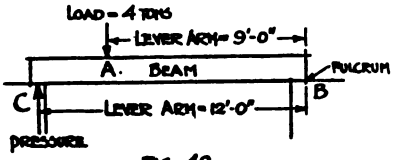

FIG. 48

which has a span of 12 ft. and carries a load of 4 tons at a point 3 ft. from the left-hand support and 9 ft. from the right-hand one.

The lever arm of the load to cause pressure at C is therefore equal to 9 ft., the lever arm of the pressure is 12 ft., and the amount of the pressure therefore equals $\frac{4 \text{ tons} \times 9 \text{ ft.}}{12 \text{ ft.}} = 3$ tons. The reaction or pressure at the left-hand abutment therefore equals 3 tons.

In order to check this the pressure at the right-hand support can also be calculated by considering the fulcrum as being at C with the load acting on the beam as a lever to cause a pressure at B (see *Fig.* 49). The pressure at B then

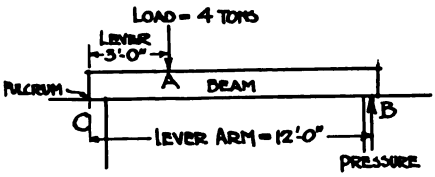

FIG. 49

equals $\frac{4 \text{ tons} \times 3 \text{ ft.}}{12 \text{ ft.}} = 1$ ton. The pressure at C equals 3 tons, the pressure at B equals 1 ton, and the total of these two is 4 tons, which is the amount of the load. In all these explanations the span of the beam is the *effective* span, which is the distance between the centres of the bearing at each end of the beam as distinct from the clear span, which is the distance between the faces of the supports. This is illustrated in *Fig.* 50,

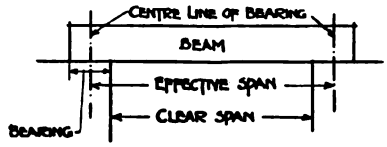

FIG 50.

and as it is the method followed in the calculations and design of reinforced concrete structures the reader should know the difference between the clear and effective spans. The pressure or reaction caused by several loads on one beam can be calculated in the same way as a single load by taking each load separately, finding the pressure caused, and then adding together all the pressures coming on one support to give the total reaction.

With distributed loads the reactions are found by considering the load as acting at its centre of gravity and calculating as before explained, but generally speaking most distributed loads are uniformly distributed over the length of the beam and when this is the case one-half of the load will be carried by each support.

These general notes on the leverage of beams as applied to the reactions at the supports should be sufficient to enable the reader to understand the general method of calculation and the importance in executing the work of giving the resistance at the supports which is called for on the drawings. It will be noticed particularly that the reaction at the support is dependent on the amount of the load and its distance from the support in relation to the effective span of the beam, and it is not dependent on the span alone. In other words, a central concentrated load of 20 tons on a beam having a span of 5 ft. only will cause a much greater reaction than a central concentrated load of 10 tons on a span of 30 feet, as the large span does not in any way increase the reaction. It must not be assumed that the bearing for a beam having a small span is of minor importance, as it may be supporting a heavy load, and the pressure on the support is independent of the actual span.

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BENDING MOMENTS.

A bending moment is a bending *amount*, and the term "moment" is used in calculations to express an amount given by the multiplying of weights or pressures by distances as distinct from a simple amount of weight alone. In the design of reinforced concrete members the designer works in pounds and inches, and while he may have an amount of 10 lbs. and a distance of 12 in. these two factors multiplied together will give an amount or *moment* of 120 inch-pounds, which means a value of weight having a leverage of distance to give a factor of force or resistance that enters into the calculations.

The bending moment is therefore a value or factor that has to be dealt with where there is a bending tendency caused by a force in pounds that has a leverage in inches, and where either one of the values, viz. the force or the leverage, is varied the bending moment is also varied. The bending moment in a beam is the value of the forces that are tending to bend and break the beam, and the resistance offered by the steel and concrete must be sufficient to overcome this tendency.

The bending tendency is caused by the opposition of the load and the reaction, as these two represent forces acting in opposite directions; the beam is the lever between the two which is called upon to carry the load to the support where the two forces can neutralize each other—to produce equilibrium. A simple example is given in *Fig.* 51, where a beam having a span of 120 in. carries a central concentrated load of 2,000 lbs. The reactions at each support in this case will be equal to 1,000 lbs. The bending tendency is caused by the load pushing downward, while the reaction resists the downward action through the medium of the beam. The theoretical action is therefore to bend the beam as shown by dotted lines where the fulcrum is indicated at the load point, and the tendency to bend is equal to the amount of the reaction multiplied by its distance from the fulcrum. In this example therefore the bending tendency is 1,000 lbs. × 60 in., which equals 60,000 in.-lbs., and this is the value of the bending moment. This is the maximum bending moment, or in other words the greatest tendency to bend, as the point considered, lettered A on the diagram, is the centre of the span and the tendency will be less if a position nearer to the support is taken.

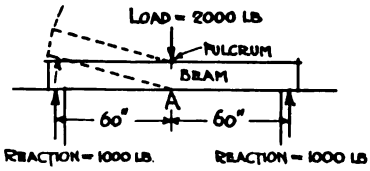

FIG. 51

In *Fig.* 52 another point marked B is shown, only 30 in. from the support; the tendency to bend here is indicated by dotted lines, and the leverage of the reaction is obviously less than the example in *Fig.* 51. The bending moment at B equals the reaction, which is still 1,000 lbs., multiplied by the distance to B which is 30 in.; therefore the value is 1,000 lbs. × 30 in., which equals 30,000 inch-pounds. It will be seen that the bending moment at B is only one-half the amount at A, and this shows that the bending moment is the greatest at the centre of the span—where a central concentrated load has to be carried—and it gradually diminishes from the centre to the support at each end.

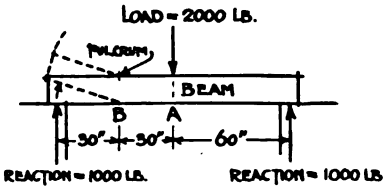

FIG. 52

This can be proved by taking a large number of points in the length of the beam and calculating the bending moment in accordance with the above method; it will be found that the variation in the moment is directly proportional to the distance of the point from the support. The bending moment at the support is nil, the maximum moment is at the centre, and the value of the moment at any intermediate point can therefore be shown graphically by setting out the span of the beam to scale; from a horizontal line joining the two supports the maximum moment is set up to scale at the centre, as shown in *Fig.* 53, and the top of the line representing this maximum

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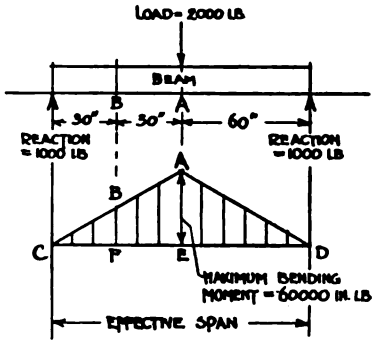

FIG. 53.

bending moment is then joined to the point where the horizontal line meets the support. This gives a triangle which is indicated by shaded lines, and the value of the vertical line inside the triangle at any point indicates the value of the bending moment at that point to the same scale as that used in setting up the value of the maximum bending moment.

For example, the bending moment at A in *Fig.* 51 is 60,000 in.-lbs., and if the scale used in setting up the bending moment diagram is, say, 10,000 in.-lbs. to 1 in., then the length of the line AE in *Fig.* 53 will equal 6 in., and the length of the line BF will be 3 in., which at 10,000 in.-lbs. per inch gives a value of 30,000 in.-lbs., as previously calculated.

It is not proposed to deal with the actual calculations for the bending moments with all kinds of loads and conditions of beams because this is beyond the scope of these notes, but it is necessary to give some definite examples of an elementary kind in order to make the notes sufficiently instructive and enable the reader to understand the difference between the various terms employed and the considerations that influence the designer when preparing the details. The bending moment is the value of the load and the reaction to cause the stresses in a beam, and the resistance of the beam is designed in accordance with the bending moment that has to be overcome. The value of the beam to resist the bending moment is known as the "moment of resistance," and therefore the moment of resistance must always equal the bending moment if the beam is to be satisfactory. It has been shown that the value of the bending moment varies at different points in the beam, and if the work is to be economical in design the moment of resistance of the beam will also vary, because it will be waste of material to provide a resistance to bending at the supports which is as great as the resistance provided at the centre where the bending moment is as a maximum. This explains why the steel reinforcement is reduced from the centre towards the supports in the case of a supported beam, because as the bending moment gradually diminishes the reinforcement can also be gradually diminished. The reduction on steel is arranged by stopping off some of the rods with hooked ends and carrying on the remainder, or by cranking up some of the rods from the tension area—where they are no longer required—to assist in the resistance to shear in the web of the beam or to provide for continuity where the beam is carried over two or more spans.

As the reinforcement is carefully calculated at the different points in a beam it is important that the work is executed accurately, as the use of rods of incorrect length may result in some of the steel being left out at a part where the bending moment is too great to justify any omission. This is particularly important in the case of concentrated loads at different points in a beam, as the maximum bending moment will then not always occur at the centre of the span, and it is impossible for the foreman to determine at sight where this maximum will occur; he is solely responsible for the execution of the work in strict accordance with the drawings.

DISTRIBUTED LOAD AND BENDING MOMENT.

Owing to the large number of cases where beams are required to carry uniformly distributed loads, it is advisable to give an indication of the variation that takes place in the bending moment at different points in the beam to avoid any impression that the foregoing examples are applicable to cases where the load is distributed and not concentrated at one point. The example in *Fig.* 54 shows a beam having a span of 120 in. which carries a uniformly distributed load of 2,000 lbs., and the maximum bending moment will be at the centre

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of the span marked A on the diagram. As previously explained, the bending

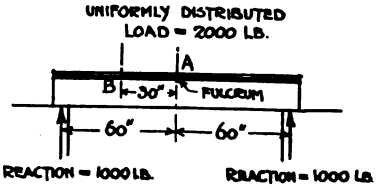

FIG. 54

moment is found by multiplying the reaction by its distance to the point at which the bending moment is required, and in this example the reaction is 1,000 lbs. and the distance to the centre is 60 ins.

The tendency to lift up the beam, as shown by dotted lines in *Fig.* 51, is, however, in this case affected by that portion of the load which is situated on the lever arm because this load acts downward and tends to counteract the upward force of the reaction.

Again it is a simple question of leverage, because the value of the load to prevent the upward tendency is the amount of the load on the lever arm multiplied by its distance to the fulcrum. As the lever arm is equal to one-half the length of the beam the proportion of the load on this length will be one-half, which is equal to 1,000 lbs., and as this will act at the centre of gravity it can be considered as acting at the point B, which is 30 in. from the fulcrum. This gives the following conditions :—

(*a*) The upward tendency of the reaction is equal to 1,000 lbs. × 60 in. = 60,000 in.-lbs.

(*b*) The downward tendency of the load on the lever arm is 1,000 lbs. × 30 in. = 30,000 in.-lbs.

As the value of (*a*) is partly neutralised by (*b*) the actual bending moment is 60,000 in.-lbs. minus 30,000 in.-lbs., which equals 30,000 in.-lbs. This is the maximum bending moment, and it will be noticed that the value is one-half the bending moment caused by the same load concentrated at the centre as given in *Fig.* 51.

The bending moment at any intermediate point will be found in a similar manner by multiplying the reaction by the distance to this point and deducting the value of the load on the lever arm multiplied by its distance to the same point.

As an example the same beam and the same loading as before are shown in *Fig.* 55, and we will assume that the bending moment at the point B is required.

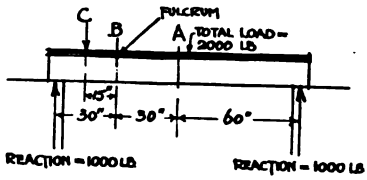

FIG. 55

The reaction is still 1,000 lbs., but the lever arm is reduced to 30 in.; the upward tendency is therefore 1,000 lbs. × 30 in. = 30,000 in.-lbs. The load on the lever arm equals one-quarter the total load, and is therefore 500 lbs., and this will act at its centre of gravity, which is 15 in. from the fulcrum. The downward tendency is therefore 500 lbs. × 15 in. = 7,500 in.-lbs. The actual bending moment is therefore 30,000 in.-lbs. minus 7,500 in.-lbs., which equals 22,500 in.-lbs. It will be noticed that this is more than one-half the bending moment at the centre, and in this respect it is different from the condition given for the bending moment at the point B in *Fig.* 52, which was exactly one-half that at the centre of the span where the load was a central concentrated one.

The variation in the bending moment

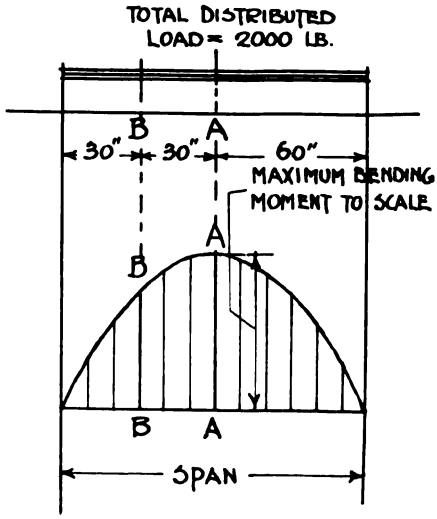

FIG. 56

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in the case of a distributed load is therefore not the same as that with a concentrated load, owing to the effect of the load on the arm of the lever. The variation in the case of a concentrated load is given by a plain sloping line, as illustrated in *Fig.* 53, whereas the variation in the case of a uniformly distributed load is given by a curved line as shown in *Fig.* 56. This curve is a parabolic curve where the maximum bending moment at the centre is set up to scale as previously explained, and the value of the bending moment at any intermediate point can be found by measuring the length of the vertical line to the same scale as before mentioned. If the curve is accurately drawn and the value at A is 30,000 in.-lbs., then the length of the line at B will represent a bending moment of 22,500 in.-lbs., as calculated. The reader is advised to work out and draw the diagrams for a few simple cases in order to become familiar with the principles and methods of working.

(*To be continued.*)

Concrete Bear Dens.

At the largest public park in St. Louis, Mo., five large bear dens have been constructed of concrete. The total surface of area of imitation rock is 75,000 sq. ft., each den having a 75-ft. frontage. The concrete was made to match the highlands along the Mississippi River south of St. Louis, more than 3,000 special casts and models being taken for the purpose.

The impressions and models were reinforced and set up in the desired positions, and cement guns were then used to spray a mixture of sand and cement over the patterns under a pressure of 40 lbs. per sq. in. The "rocks" were made from 2 to 10 in. thick, according to the load which they had to support.

Initially, 42 impressions were made of the river bluffs, each impression covering 30 ft. square. The casts were made by coating the surface of the natural rocks with sterin oil and then applying a covering of moulding plaster, ½ in. thick, after the area had been fenced in by the necessary plaster frames. Braces and reinforcements were provided so that the casts could be removed after the plaster had set. Gelatin impressions were then made from the original casts, and these impressions, the basis for the construction of the permanent patterns, were built and transported to St. Louis, where they formed the background upon which the concrete coating was sprayed.

Concrete Bear Dens at St. Louis.

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Thames ballast	,,	10	6
Broken brick (¾ in.)	,,	11	6
Best British Portland Cement	per ton 58s. to	63	0
"Ferrocrete" Portland Cement	delivered London 10s. per ton extra		
"Super-Cement"	per ton 88s. to	93	0
"Lightning" Brand Aluminous Cement	,, 100/- f.o.r. works.		

BOARDING FOR SHUTTERING—		Sawn. s.	Sawn. d.	Wrot. s.	Wrot. d.
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in. . . . . from £23 per standard
3 in. by 6 in. and 3 in. by 7 in. . . . . ,, £23 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	0
9/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds	,,	14	6
¼ in. Rounds	,,	15	6

Breeze Slabs per yd. super: 2 in., 1/11; 2½ in., 2/4; 3 in., 2/9; 4 in., 3/6.

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 : 2 : 4—		s.	d.
Do. do. in foundation	per yard cube	48	6
Do. do. in columns	,, ,,	53	6
Do. do. in beams	,, ,,	53	6
Do. do. in floor slabs 4 in. thick	per yard super	5	8
Do. do. in floor slabs 6 in. thick	,, ,,	8	5
Do. do. in floor slabs 9 in. thick	,, ,,	12	3
Do. do. in walls 6 in. thick	,, ,,	8	8

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—		s.	d.
From ¼ in. to ⅝ in.	per cwt.	25	0
,, 9/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2⅞ in.	,,	23	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1¼*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2¼*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.* (per bend per cwt.).

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.* (per bend per cwt.).

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	,, ,,	1	1½
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1½
Do. do. as last, in narrow widths	,, ,,	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Carpenters working on old shuttering, 1/10½; Labourers on building works, 1/4½; Men on mixers and hoists, 1/5½; Bar-benders, 1/5½.

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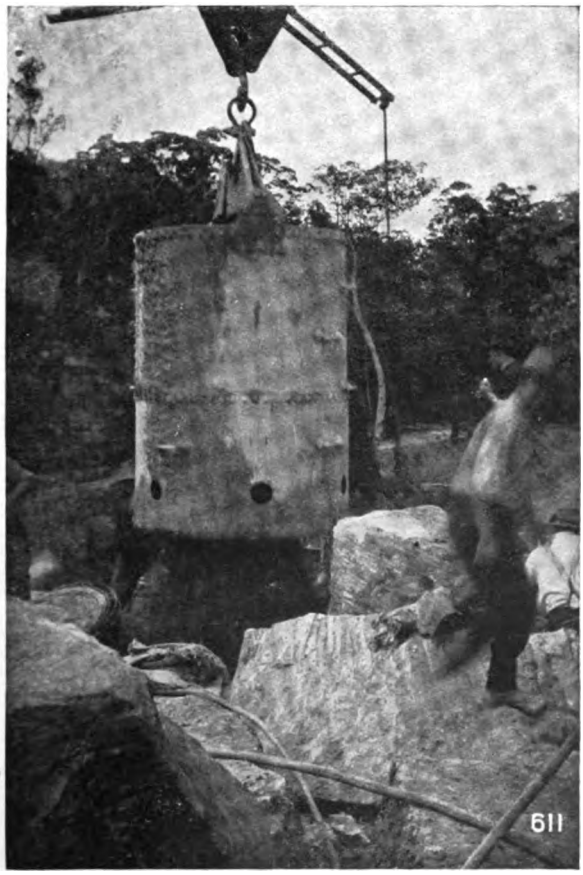
New Concrete Placing Plant.

A NEW type of plant for placing concrete has been put on the market by Messrs. Ransome Machinery Co. (1920), Ltd., of 14-16, Grosvenor Gardens, S.W.1, in the form of a skip which discharges through a valve in the bottom. The skip is specially adapted for use with cableways, and by the operation of the valve the whole or any desired quantity of the concrete can be discharged and placed in position where required. The skip consists of a cylindrical body or shell, the bottom portion of which acts as a protecting wall for the discharging valve. This conical valve is held into the open base of the concrete retainer, and operated by a rod running up the centre of the shell and passing through a bearing at the top. Above this bearing the rod is jointed and connected to a crank on a worm-wheel, which is operated by a worm on a shaft, the latter in turn carrying a chain wheel and chain outside the shell. The valve is opened or closed by means of this outside chain. The whole operating mechanism is supported on steel bearers across the open top of the shell and protected by a sheet steel cover. Grips are provided outside the shell for guiding the skip into position before discharging. This positive opening and closing of the valve, in relation to the body of the skip, gives absolute control of the discharge, and the rate of discharge of the contents. Fur-



New Concrete Placing Plant.

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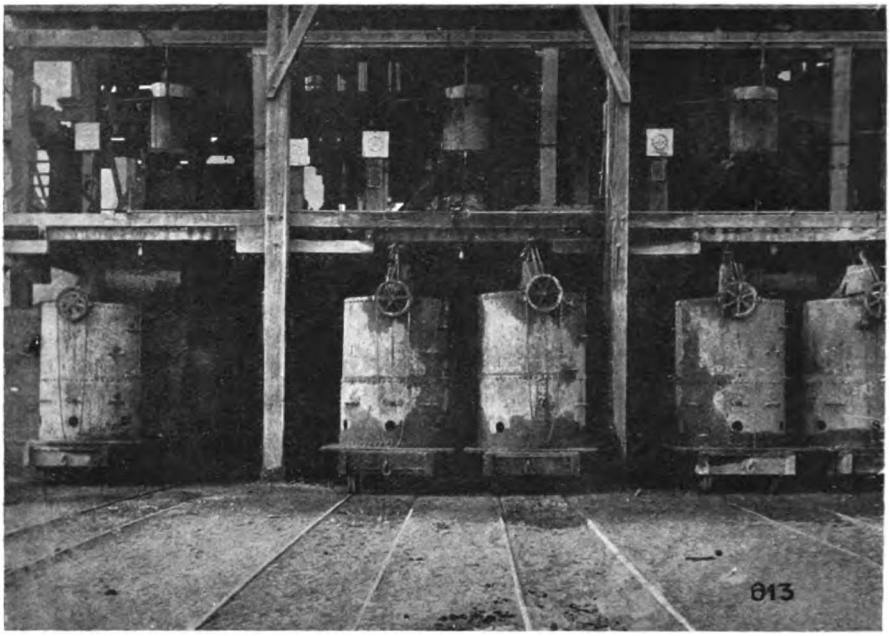

New Concrete Placing Plant.

thermore, this control is quite independent of whether the skip is suspended or resting on the ground, as the valve and the base of the container are fully protected by the bottom portion of shell. Inspection orifices are provided in the latter so that the working of the valve may be observed. The container is watertight, and there can be no leakage of cement and consequent weakening of the mixture, and wear on the valve and seating is automatically taken up as it occurs.

Owing to the use of valves which can be controlled the flow of concrete may be regulated so that it is placed practically without shock and without injuring the forms. Our illustrations show one of these skips in use discharging into a chute and a battery of skips being charged direct from the mixers; when charged in this way they are run under the mixers on a flat bogie. We are informed that these skips have been successfully used by a Colonial government in the construction of two large dams, and that the saving effected compared with the method of placing in use before these skips were introduced on the work amounts to over 41 per cent., or 5*s*. per cu. yd. of finished concrete. The average time for the release of concrete from a 3-cu. yd. skip is 15 seconds.

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Prospective New Concrete Work.

AIRDRIE.—*Street.*—A street is to be constructed from Chapel Street to Mack Street.

ALLOA.—*Bridge.*—The T.C. is inquiring into the cost of constructing a bridge across the river.

ANNFIELD.—*Houses.*—The Housing Committee has recommended the U.D.C. to seek permission for the erection of 103 houses.

BARRY.—*Sea Wall.*—The T.C.'s improvement scheme includes the construction of a sea wall across Whitmore Bay.

BELFAST.—*Houses.*—The Corporation is seeking powers for a loan of £500,000 for housing purposes.

BLAYDON.—*Houses.*—The U.D.C. has applied for permission to build 182 houses at Bleach Green.

BLYTH.—*Reservoir.*—The T.C. has a scheme in hand for the construction of a reservoir at Horton.

BRENTFORD.—*Bridge.*—A scheme for the improvement of Gunnersbury Lane includes the construction of a bridge over the railway.

DERBY.—*Roads.*—The T.C. has asked for a loan of £36,000 for the construction of roads on the Cowsley, Newdigate, and Osmaston housing estates.

FROME.—*Houses.*—The R.D.C. proposes to erect 146 houses in the next two years.

GIRVAN.—*Harbour Wall.*—A scheme for the construction of a wall at the harbour is being considered by the T.C. and the R.C.

GLANRANNELL. — *Footbridge.* — The Llandovery R.D.C. has submitted to the M.H. a scheme for the construction of a footbridge at Glanrannell.

GLOUCESTER.—*Houses.*—The T.C. proposes to erect 100 houses.

GREAT YARMOUTH. — *Houses.* — The Corporation's housing scheme provides for the erection of 382 houses.

HIGH WYCOMBE.—*Houses.*—The D.C. has purchased land for the erection of 100 houses.

(*Continued on next page.*)

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LEWISHAM (*London*).—*Houses.*—The B.C. proposes to erect 150 houses.

LLANDUDNO.—*Sea Defence Work.*—The U.D.C. has applied for a loan of £3,203 for sea defence works.

LONGBENTON.—*Houses.*—The U.D.C. has approached the M.H for sanction to erect 56 houses.

LOWESTOFT.—*Playgrounds.*—The Education Committee proposes to spend £2,500 for concreting playgrounds.

LUTON.—*Houses.*—The T.C. proposes to erect 100 houses.

MAIDENHEAD.—*Concrete Bridge.*—The Borough Surveyor has submitted plans to the M.T. for the erection of a reinforced concrete bridge.

MANCHESTER.—*Concrete Houses.*—The Corporation is considering a scheme for the construction of concrete houses.

MICKLEBRING.—*Houses.*—The Markham Main Colliery Co. proposes to erect a colliery village with accommodation for about 5,000 people.

MOLESEY.—*Concrete Piles.*—The U.D.C. proposes to erect concrete piles along the River Mole at a cost of £355.

MONTROSE.—*Concrete Bridge.*—A suggestion has been adopted for an inquiry into the cost of a reinforced concrete bridge in place of the existing "Chain Brig."

MORETON. — *Houses.* — The Wirral R.D.C. proposes to build 660 houses.

NOTTINGHAM.—*Concrete Bridge.*—A reinforced concrete bridge is to be erected over the Trent at a cost of about £120,000.

OULTON.—*Bridge.*—The D.C. is considering a scheme for the construction of a bridge across the Leeds Ring Road at Oulton.

POOLE.—*Footbridge.*—The B.C. proposes to erect a footbridge between Poole and Hanworthy, estimated to cost £3,000.

PRESTON.—*Concrete Houses.*—At a cost of about £100,000 the T.C. proposes to provide for the erection of 200 concrete houses.

RAMSGATE. — *Houses.* — The Corporation is negotiating for the purchase of land on which to erect 400 houses.

REDCAR.—*Road.*—The Corporation has a scheme in hand for the construction of a road from Park Avenue to Thrush Road.

REDCAR.—*Sewage Disposal Works.*—The Corporation has applied for a loan of £10,000 for the construction of sewage disposal works.

RENFREW.—*Quay Wall.*—The T.C. is negotiating for the construction of a quay wall at Renfrew.

SCARBOROUGH.—*Sea Wall.*—The Corporation proposes to carry out alterations to the Royal Abert Drive sea wall.

SIDMOUTH.—*Sea Defence Works.*—The U.D.C. has applied for permission to borrow £3,000 for sea defence works.

SOUTHWOLD.—*Concrete Bridge.*—The T.C. proposes to construct a reinforced concrete bridge at an estimated cost of £4,000.

STAVELEY. — *Sewage Works.* — The Chesterfield R.D.C. has submitted a scheme to the M.H. for a joint sewage disposal works for the surrounding parishes.

STIFFORD (*Essex*).—*Bridges.*—A sum of £15,800 is to be spent by the Essex C.C. on the construction of bridges at Stifford, Stock Wash, Hoppit, and Causeway.

SUDBURY.—*Concrete Road.*—The Wembley U.D.C. proposes to improve the corner of Elms Lane by the construction of a 16ft. concrete carriageway.

TAUNTON.—*Houses.*—The T.C. is seeking power to appropriate land adjoining Lambrook Road as a site for the erection of 100 houses.

THORNE.—*Sewage Disposal Works.*—The R.D.C. has applied to the M.H. for sanction to borrow £32,000 for sewage disposal works.

WATERFORD.—*Sea Wall.*—Improvements to the sea wall are to be made at a cost of £1,000.

WATERTON.—*Road.*—A by-pass road is to be constructed from Waterton to Laleston, at a cost of £10,000.

WEDNESBURY.—*Houses.*—The Corporation has applied for a loan of £12,936 for the erection of 30 houses.

WELSHPOOL.—*Bridge.*—The T.C. and the Montgomery C.C. propose to erect a bridge over the main road at Buttington.

WHITEHAVEN. — *Houses.* — The T.C. proposes to build 100 houses.

WIGTOWNSHIRE.—*Sea Wall.*—The Surveyor has submitted a scheme for the construction of a reinforced concrete sea wall on the Stranraer-Drunmore Road, estimated to cost £2,800.

YORK.—*Concrete Houses.*—The Corporation proposes to erect four experimental concrete houses.

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Tenders Accepted.

BLACKBURN.—*Bridge.*—The T.C. has accepted the tender of Messrs. Gray's Ferro-Concrete Co., Glasgow, for the construction of a reinforced concrete bridge over the canal at Whiteburk.

CREWE.—*Footbridges.*—The T.C. has accepted the tender of Messrs. Clegg Bros., Burnley, at £1,455, for the construction of three reinforced concrete footbridges.

CROYDON.—*Concrete Wall.*—The T.C. has accepted the tender of Messrs. G. E. Everitt & Sons, Ltd., Croydon, at £2,254, for the construction of a concrete wall at Mitcham Road cemetery.

EASTBOURNE.—*Concrete Houses.*—The Corporation has accepted the tender of Mr. J. Knox for the erection of 200 concrete houses.

FILEY (*Yorks.*).—*Concrete Roads.*—The U.D.C. has accepted the tender of Messrs. B. Fell & Sons, at £562, for the construction of two reinforced concrete roads with an approximate area of 1,200 super yds.

GLASGOW.—*Terracing.*—The Corporation has accepted the tender of Mr. J. Drysdale, Glasgow, at £877 os. 8*d.*, for the construction of concrete terracing round the bandstand in Bellahouston Park.

HASLEMERE.—*Channel.*—The U.D.C. has accepted the tender of Messrs. A. Streeter & Co., at £875, for the construction of a 200-ft. concrete channel at Sturt Road.

HASTINGS.—*Grand Stand.*—The T.C. has accepted the tender of Messrs. H. & J. Tayler, at £5,300, for the construction of a reinforced concrete stand at Pilot's Field.

HULL.—*Concrete Houses.*—The Corporation has accepted the tender of Messrs. Con Greenwood & Sons, at £476 10*s.* per house, for the erection of 200 concrete houses.

NEWCASTLE.—*Concrete Houses.*—The T.C. has accepted the tenders of Mr. A. Anderson, at £76,536, for the construction of 150 concrete houses, and Mr. J. S. Hetherington, at £151,056 12*s.* 11*d.*, for the construction of 300 concrete houses.

NEW ROMNEY.—*Concrete Kerbing.*—The T.C. has accepted the tender of Mr. J. R. Philcox for the supply of concrete kerbing at 3*s.* per yd.

SPALDING.—*Bridge.*—The T.C. has accepted the tender of Messrs. Purdie Lumsden & Co., Ltd., for the construction of a reinforced concrete bridge on the Spalding-Stamford main road.

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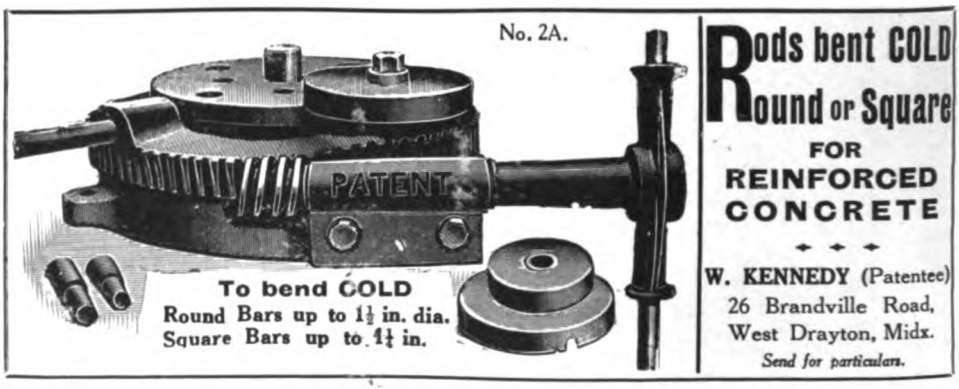
Trade Notices.

THE very large range of products which may be produced on the "Hawthorne" concrete roofing tile machine is well illustrated in a catalogue issued by the Concrete Tile Machinery Co., of Illinois, whose London agents are Messrs. John F. Allen & Co., of 49, Leadenhall Street, E.C.3. The machine is essentially made for the mass production of concrete tiles under factory conditions, and will, we are informed, turn out forty squares of tiles per day at a total cost of about 22*s*. per square.

The frame of the machine is made of heavy castings securely put together to avoid vibration. All gears are enclosed and run in oil, all moving parts are protected so that concrete cannot work in. Stamped steel pallets are fed into the machine by one operator at the rate of fourteen per minute, and as they are carried forward they are automatically covered by a thin coat of oil. The pallet then passes under a hopper which has an adjustable double gate that controls the amount of concrete carried out on the pallet. The concrete is discharged from the mixer into a feeder which keeps the correct amount of concrete of uniform density in the hopper at all times. As the pallet filled with concrete passes out of the hopper, the concrete is packed and the top surface formed by a tamping device which strikes sixty blows of uniform pressure to each tile. After leaving the tamper the tile goes under the colour feeder, consisting of a double set of full floating trowels, between which a top coating of cement, colour pigment, fine sand and water, mixed just thick enough to flow by its own weight, is applied. The first trowel smooths the concrete and the second the colour coating. From the colour feeder the tile is automatically transferred to a belt conveyor that carries it to the curing space.

Change of Address.

Messrs. Stewart & McDonnell, engineers and public works contractors, have removed from Cowley Street, S.W.1, to King's Buildings, Smith Square, Westminster, S.W.1 (Telephone: Victoria 4444).



BOOKS ON CONCRETE.

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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EDITORIAL NOTES.

The Fineness of Portland Cement.

THE days when the fine grinding of cement can be used as a "talking point" by cement salesmen are coming to an end. It is being more and more recognised that the only desiderata of cement are strength, endurance, and suitable setting qualities, and so long as these are provided by the manufacturer the means by which they are obtained—of which fine grinding is only one—are of little interest to the buyer. There is, of course, a definite requirement for fineness in most cement specifications, but it should be remembered that the scientific development of cement manufacture is recent and until lately it has been desirable for standard specifications for cement to safeguard the buyer by ensuring that the cement shall be produced by certain well-tried methods proved by experience to be safe and reliable. The early days of cement manufacture had shown that coarsely-ground cement was liable to be weak and unsound, and consequently

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fine grinding was stipulated in specifications. Both manufacture and specifications were, in fact, empirical. With increasing knowledge it is probable that specifications will abandon all attempts to regulate the process of manufacture, leaving the producer a free hand to obtain the specified strength and stability in the manner most convenient to him.

It would therefore be a retrograde step to adopt the suggestion given prominence by one of our engineering contemporaries that the present method of ascertaining the fineness of cement by sieve tests should be superseded by the use of an apparatus for determining the proportion of "flour" in the cement. The objections raised to the sieve test are that it is unreliable because of the difficulty of obtaining standard and consistent wire mesh for sieves, and that it fails to discriminate between the active and inert particles in a cement. It is suggested that an appliance such as the "flourometer" should be adopted, and cement specifications would presumably require a minimum proportion of "flour."

Such a recommendation as this is objectionable primarily because it constitutes a further interference with the cement manufacturer, or, at the best, substitutes one means of dictation for another; and such interference is becoming unnecessary. There are, however, other reasons why the "flour" test should not be introduced into cement specifications, and one of these illustrates the disadvantages occurring when specifications are empirical. The "flour" test is one in which the fine and coarse particles in cement are separated by elutriation either by air or by a suitable liquid. By passing a current of air or liquid through a given quantity of cement for a certain period the fine particles of flour are carried away and the coarse particles remain and can be weighed to determine their proportion in relation to the whole. It is only the flour that has cementitious properties at early dates; the coarser particles do not contribute their quota to the strength of cement until later, and indeed some of the larger grains may be permanently inert. Hence a cement with a low proportion of flour is weak at early dates, and, conversely, when the proportion of flour is high the cement generally develops its strength rapidly. A cement containing 100 per cent. flour would probably reach its maximum strength when neat at seven days, and by so doing would fail to comply with the British Standard Specification which requires a proportionate increase from seven to twenty-eight days. Hence our contemporary's desire for more flour in cement is incompatible with the specification requirement. The origin of this stipulation for increase in neat tensile tests from seven to twenty-eight days dates from the time when cement was crudely manufactured, and any failure to grow in strength was frequently a sign of impending disintegration. Although the test still remains, the necessity for it has disappeared because the stability of cement can now be ascertained by quicker and more adequate tests.

It has already been mentioned that fine grinding is only one of the means of obtaining strength and stability in cement, and to describe the fineness test as "one of the most important and necessary" is an undesirable emphasis. The advantages obtainable from fine grinding of cement can also be produced by changes in chemical composition, by harder burning, or by finer grinding and more intimate mixing of the raw materials. The manufacturer selects from these possibilities those which best serve his purpose to produce a cement of given strength and stability, and it would be as illogical for the buyer to demand an

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extra percentage of lime or alumina in the cement as it is to insist upon an extra percentage of flour. The tests for strength and stability of cement should be as stringent as the best brands can stand, but the manufacturer cannot give the best and most economical results if he is unduly restricted in methods of manufacture. We take leave to doubt whether " the flour question would be a vital one " in connection with the grouting work of St. Paul's Cathedral. If the cracks and interstices in the rubble within the piers were of such dimensions that grains of cement retained on a sieve of 180 meshes per linear inch could not enter, then the necessity for grouting could hardly have arisen, and when dealing with neat cement it must be remembered that the finer the cement the greater is the proportion of water required to produce a given consistency, and the greater the proportion of water the greater is the tendency to contraction.

Effect of Molten Sulphur upon Concrete.

THE American technical press has recently contained references to the effect of immersing cement mortar in molten sulphur. Such a treatment of cement mortar would be regarded as an absurdity by anyone versed in the chemistry of cement and concrete, and presumably the idea for the treatment arose among those whose interests were with sulphur rather than with cement and concrete. The results as reported from America are, however, quite unexpected. A weak mixture of cement and sand with a tensile strength of about 200 lbs. per sq. in. had its tensile strength increased to as much as 2,000 lbs. per sq. in. after immersion in molten sulphur. These tests have been repeated in England and the American results are confirmed. It is found that the tensile strength of neat cement is doubled by immersion for a few hours in molten sulphur, and briquettes made of 3 parts sand to 1 part cement gain a fourfold strength by the same treatment. Compression tests show the same characteristics, and results of 15,000 lbs. per sq. in. for 3 to 1 mortar in compression have been obtained from the sulphur-immersed test pieces. The reason for the enormous growth in strength has apparently not been established, nor is it stated whether the effect is permanent.

So far as is known there has been no attempt at a practical application of the discovery, and there are obvious difficulties because it is necessary for the sulphur to penetrate the interior of the mortar to obtain the results quoted and not merely to remain as a superficial layer. The possibilities of quadrupling the strength of mortar (and presumably concrete) are of such potential value that the matter should not be left without attempts to take practical advantage of its application.

Expanded Concrete.

UNDER ordinary circumstances the aim of the concreter is to produce concrete of maximum density, because by this means the best use is made of the most expensive ingredient of the concrete, namely cement, it being understood that strength is the principal quality required. But there are occasions when other qualities must predominate. An example is given in an article in this issue. The aim is to make concrete of minimum density for heat-insulating properties, and this is done in a very ingenious fashion. We do not endorse all the views expressed in the article, but the possibility of producing concrete weighing as little as 20 lbs. per cubic foot may have interesting developments.

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Fig. 1.—Pit-Head Gears in Reinforced Concrete.
(*See p.* 303.)

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Pit-Head Gears in Reinforced Concrete.

By W. L. Scott, A.M.Inst.C.E.

THERE are few branches of engineering or building which give more structural scope for the use of reinforced concrete than pit-head gears and the adjacent receiving platforms, screening and tipping sheds, etc. In considering the economical question, it is necessary to have regard to the requisite life of such works. The deterioration of timber exposed to the weather and the frequent painting necessary to prevent the corrosion of steel head-gears frequently enable reinforced concrete construction to compete with these materials. The ordinary steel frame and brick building construction is already being successfully competed with by reinforced concrete both as regards cost and speed of erection.

In Great Britain little reinforced concrete work has so far been done by the colliery companies, either for new work or for renewals, and construction in steel, timber and brick work is still almost exclusively employed. Continental countries, however, particularly France, have, in the almost complete reconstruction of their mining districts, employed reinforced concrete to a surprising extent, and this is to be seen everywhere for bridges, railway stations, goods sheds, power stations, and mine buildings. Pit-head gears of all kinds have been erected in France, principally in the Bethune and Lens districts, and a selection of these, together with some of the adjacent buildings, is given in *Figs.* 1 to 6.

In some of the earlier pit-head gears concrete is only employed for the inclined and vertical back struts, head platforms, and supports, structural steel work being utilised for the construction of the tower itself. This arrangement is shown in *Figs.* 3, 4, and 5. *Fig.* 4 shows the head-wheel structure standing alone, and *Fig.* 5 shows the same headgear with surrounding buildings complete and the steel cage-towers in position. After these gears had been working for some time it was found that there was no appreciable vibration, and in subsequent works the complete headgear was constructed in reinforced concrete as shown in *Figs.* 1 and 6.

The isolated headgear illustrated in *Fig.* 1 is situated at the No. 1 pit of the Lens Mine Co. Three of these gears of identical type have been erected for this company. The total height from the ground to the roof of the tower is 111 ft. 6 in. The pits over which these headgears are erected were demolished during the war and the concrete headgears were erected for the preliminary purpose of pumping out and excavating the pits. In the completed work they are surrounded by buildings, and the enclosed cabin illustrated in *Fig.* 1 has been introduced to facilitate the sealing of the roof to the future building at the points where the towers penetrate it. *Fig.* 2 shows one of these headgears during construction.

Like all other engineering developments, improvements have been made to headgears both regarding their efficiency and appearance since reinforced concrete was first employed for their construction. *Fig.* 3 illustrates one of the earlier headgears erected in this material, and compared with that given in *Fig.* 1, this being one of the latest types, the improvement can be seen.

A point of interest regarding the design of these headgears is the fact that

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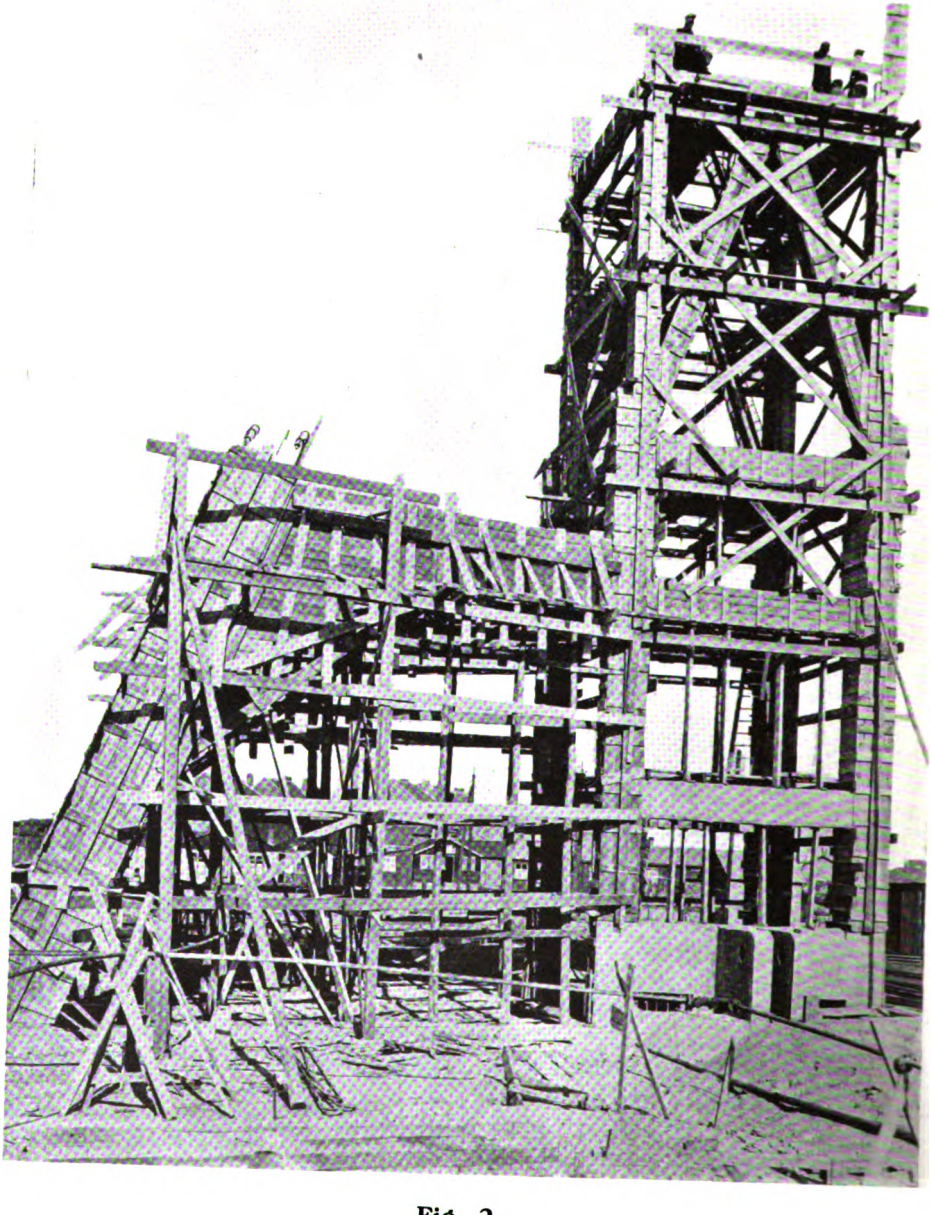

Fig. 2.

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they are arranged in a manner suited to the material of which they are composed. Many structures now constructed in reinforced concrete show marked evidence of having been copied from earlier works serving the same purpose but built of steel or timber, or a combination of both. This not only applies to the structural lay-out but also to the architectural treatment, and it is only recently that the engineering profession, led by architects of initiative, is realising that reinforced concrete, owing to its nature and to the structural adaptability obtainable by its employment, requires to be treated from a new standpoint. Many industrial structures, included amongst which are pit-head gears, have in the past been notoriously hideous and in this direction the scope for the proper architectural treatment in reinforced concrete is most wide.

Fig. 3

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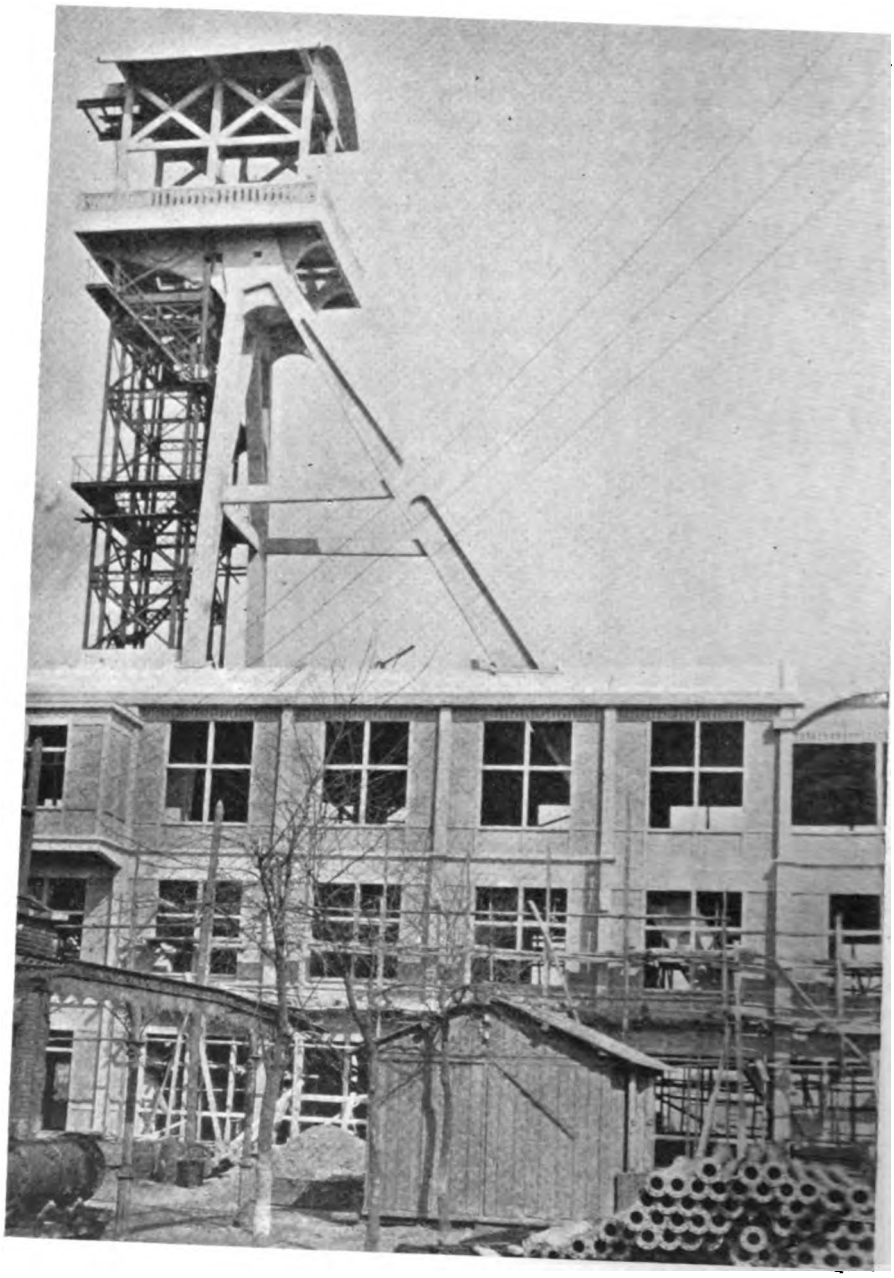

Fig. 5.

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The advantages of reinforced concrete over other constructional material is most marked in structures which are primarily subjected to compression, and in pit-head gears it will be seen that this is almost exclusively the case. Some of these towers are of considerable height, amounting in some cases to 100 ft., and in these cases the design of the inclined back stays requires special treatment. The assumed direct thrust coming upon them is taken at a much greater figure than that ordinarily produced, *i.e.* the breaking load of the ropes. In addition, since they are unsupported in the vertical direction along their horizontal span, the bending moment induced by their own weight requires to be taken into account. These moments, of course, vary according to the inclination of the members, and the requisite stiffness against buckling depends upon their length from the ground to the head towers.

The most economical cross-section is found to be a "T," and the profile

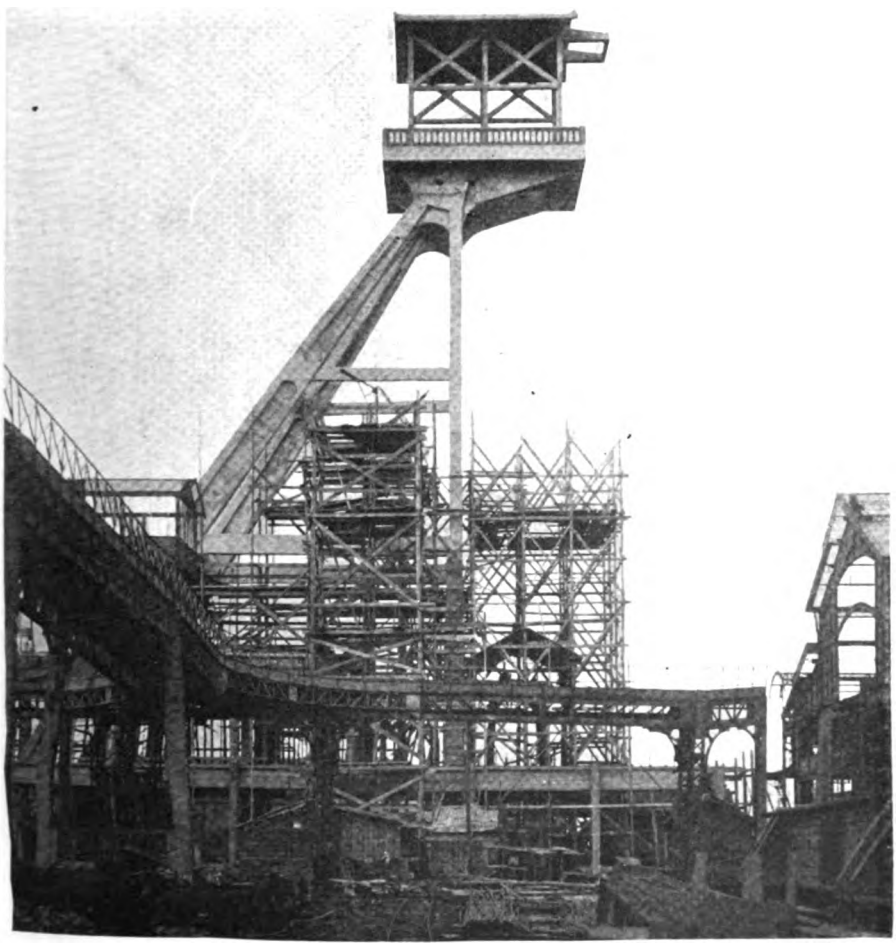

Fig. 4.

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is arranged with a slight camber on the upper face in such a way that under the maximum working forces and moments the centroid of the section is coincident with the resultant axial compressive force at all normal cross sections.

It might be added that in work of this class structural sections coated with concrete have been found to be not only less satisfactory from the technical point of view but at the same time more costly. By the use of plain round rods the concrete can be reinforced in a very precise and simple manner, and the presence of considerable masses of useless material avoided. The whole of the structures illustrated herewith are of Considère design.

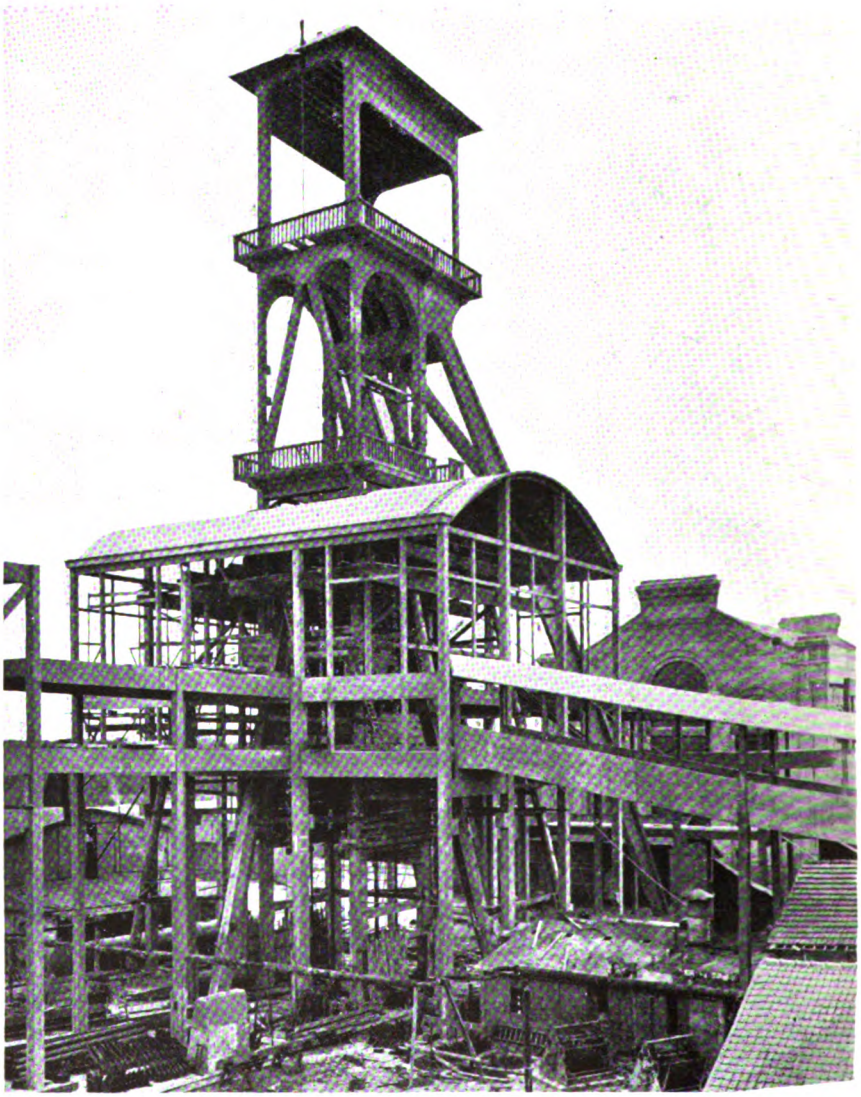

Fig. 6.

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INFLUENCE LINES APPLIED TO CONTINUOUS BEAMS.

By W. S. GRAY, B.A., M.A.I.(Dublin), A.M.Inst.C.E.I.

For a series of equal spans continuous over the supports, which is such a common arrangement in reinforced concrete construction, the method of influence lines readily gives the bending moments at the different supports due to any system of loading. The method also shows graphically the spans which must carry live load in order to produce a maximum moment at any support.

An influence line for any particular section of a structural member is a curve which shows by the variation of its ordinates how a unit force moving over the structural member stresses the particular section for which the influence line is drawn. A very simple example is given in *Fig.* 1, which is the influence line for the bending moment at the centre of a simply-supported beam due to a concentrated load at any point of the beam. It is well known that if

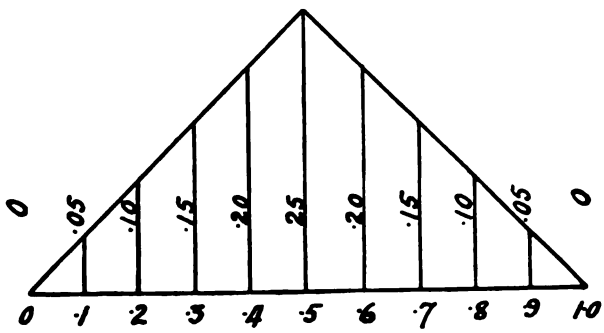

fig. I.

the load act at the centre of the beam the bending moment at the same point is $\frac{WL}{4}$, whereas if the load act directly over the support the moment at midspan is zero. If the load is acting at some intermediate point the bending-moment coefficient bears the same ratio to $\frac{1}{4}$ as the distance of the load from the abutment bears to the half-span. Therefore if we erect an ordinate at the midspan equal to 0·25 in length on a convenient scale and join the upper end of this ordinate to the two supports, and erect other ordinates at, say, the tenth points of the span, these ordinates will represent the coefficients by which WL must be multiplied to give the amount at midspan for a load at any one of the tenth points. Suppose, for example, that the load is at 0·3 of the span from the left abutment, that both span and load are unity, then the moment at midspan due to this loading is 0·15. If W lbs. is the point load and l inches is the span, then the moment at midspan is $M_c = 0{\cdot}15\ Wl$ in.-lbs.

To facilitate the reading of the lengths of the ordinates when the load does not act at one of the tenth points the influence line may be drawn on squared

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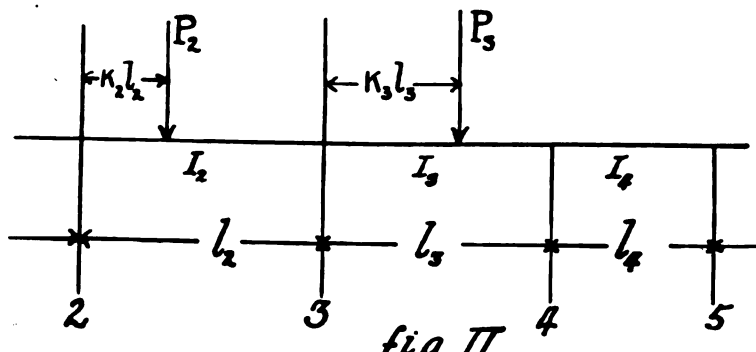

fig. II.

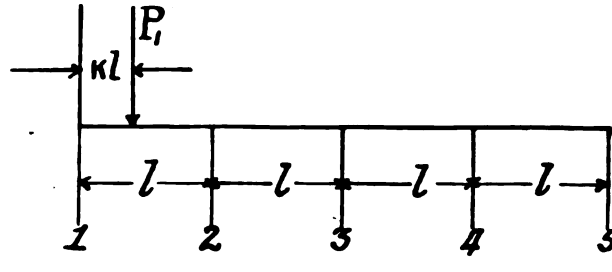

fig. III

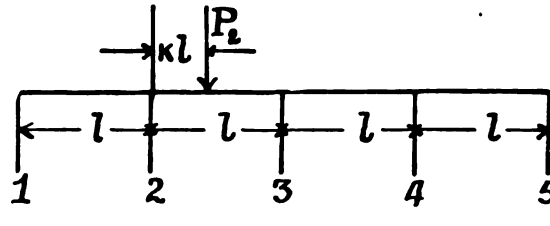

fig. IV

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paper. It is also convenient to mark on the diagram the values of the ordinates at the third points of the span. If, in a practical example, the load acts at 5 ft. 9 in. from the left-hand abutment of a 14-ft. span, the required fraction of the span is given at once by the slide-rule as 0·41, which is the point where the ordinate must be scaled.

For uniform load all over the span it is easy to show that the moment at midspan, M_c = area of influence diagram $\times\ wl^2$, where w = the uniform load per unit length of span. As a check on this statement we have,

$$M_c = 0{\cdot}5 \times 0{\cdot}25 \times wl^2 = 0{\cdot}125wl^2$$
$$= \frac{Wl}{8}, \text{ as is well known.}$$

A particular case of this is where the uniform load extends over a portion only of the span; here M_c is found by multiplying wl^2 by the area of the influence diagram intercepted between the ordinates at the two ends of the load.

Uniformly varying loads may be divided into closely-spaced point loads of increasing intensity and treated as concentrated loads.

The derivation of moment influence lines for continuous beams will now be made, using the Theorem of Three Moments, which, in its most general form for supports all on the same level, may be expressed as (*see Fig.* 2),

$$M_2l_2I_3 + 2M_3(l_2I_3 + l_3I_2) + M_4l_3I_2 =$$
$$- \Sigma P_2l_2^2I_3(K_2 - K_2^3) - \Sigma P_3l_3^2I_2(2K_3 - 3K_3^2 + K_3^3)$$

If I = constant, then we have,

$$M_2l_2 + 2M_3(l_2 + l_3) + M_4l_3 = - \Sigma P_2l_2^2(K_2 - K_2^3) - \Sigma P_3l_3^2(2K_3 - 3K_3^2 + K_3^3)$$

and if l = constant,

$$M_2 + 4M_3 + M_4 = - \Sigma P_2l(K_2 - K_2^3) - \Sigma P_3l(2K_3 - 3K_3^2 + K_3^3).$$

For any three adjacent supports such an equation may be written, and further simplification results from the fact that if there is no load on the right-hand span of the two spans under consideration then the second term on the right-hand side of the last equation vanishes.

Let us now consider a beam of four equal spans (*Fig.* 3) with the extreme ends simply supported and a point load P_1 acting on the span 1–2 at a distance Kl to the right of the left end of the beam. Then, by the Theorem of Three Moments,

$$M_1 + 4M_2 + M_3 = - P_1l(K - K^3),$$

but $M_1 = 0$ since the left end is simply supported.

$$\therefore\ 4M_2 + M_3 = - P_1l(K - K^3) \qquad . \quad . \quad . \quad \text{(A)}$$

Also, $M_2 + 4M_3 + M_4 = 0$, since there is no load on either span 2–3 or 3–4 (B)

Similarly, $M_3 + 4M_4 + M_5 = 0$

but $M_5 = 0$.

$$\therefore\ M_3 + 4M_4 = 0 \qquad . \quad . \quad . \quad . \quad . \quad \text{(C)}$$

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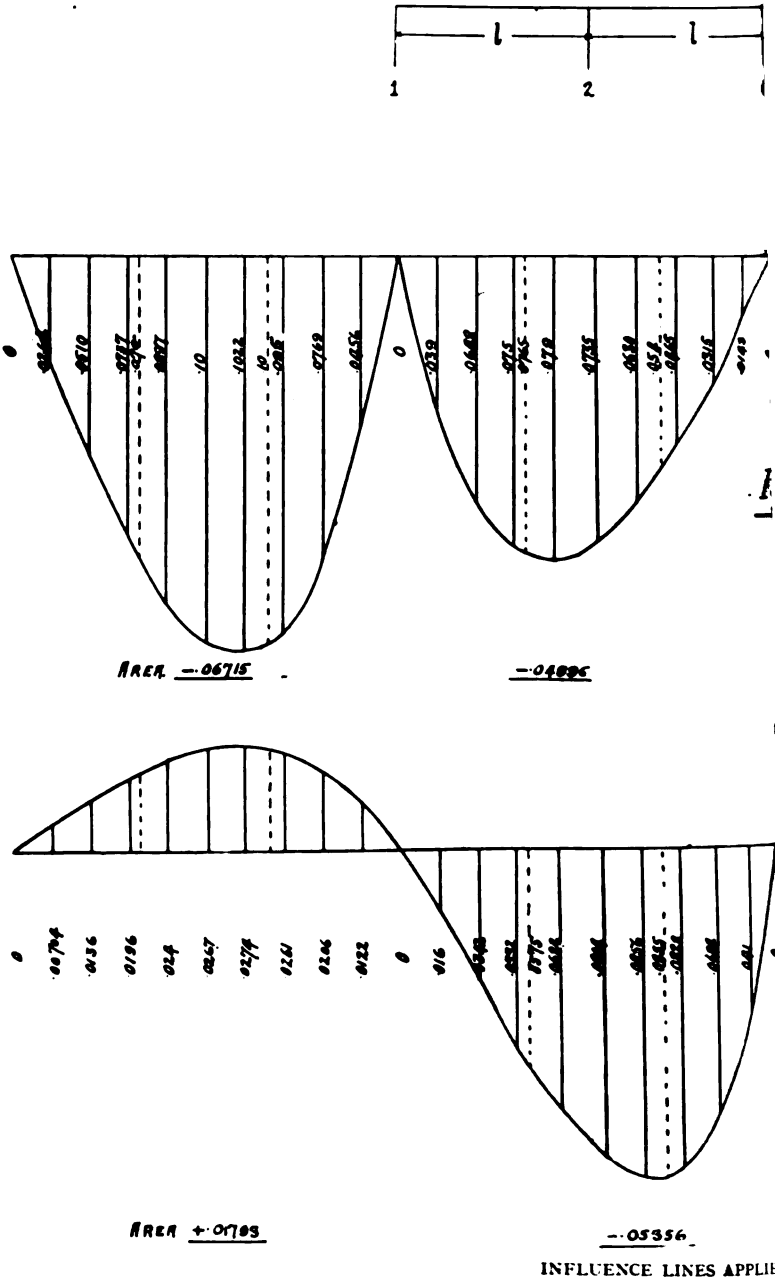

INFLUENCE LINES APPLIE

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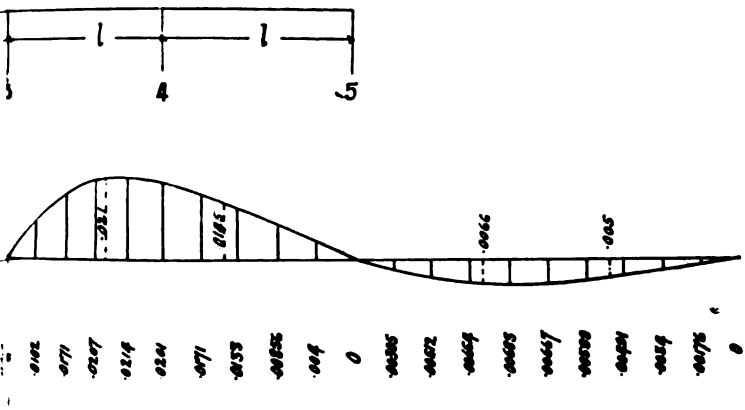
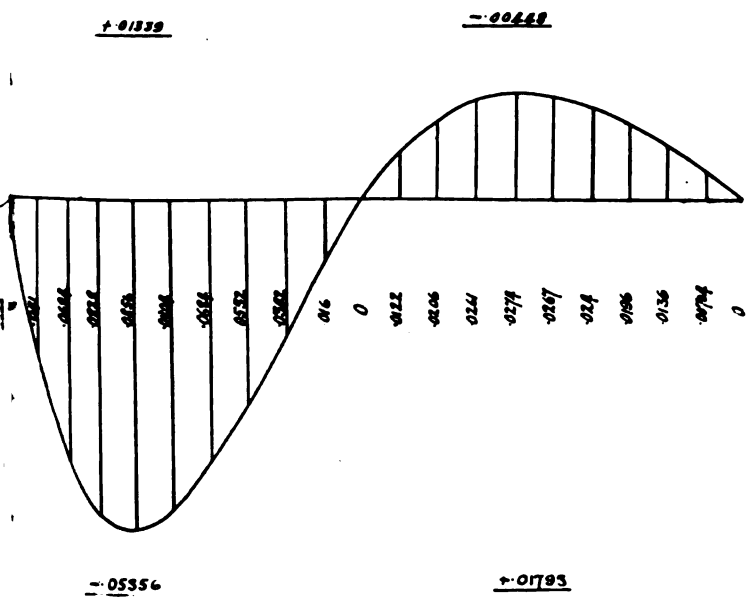


Influence Lines for M_2 and M_3

fig. V



) TO CONTINUOUS BEAMS.

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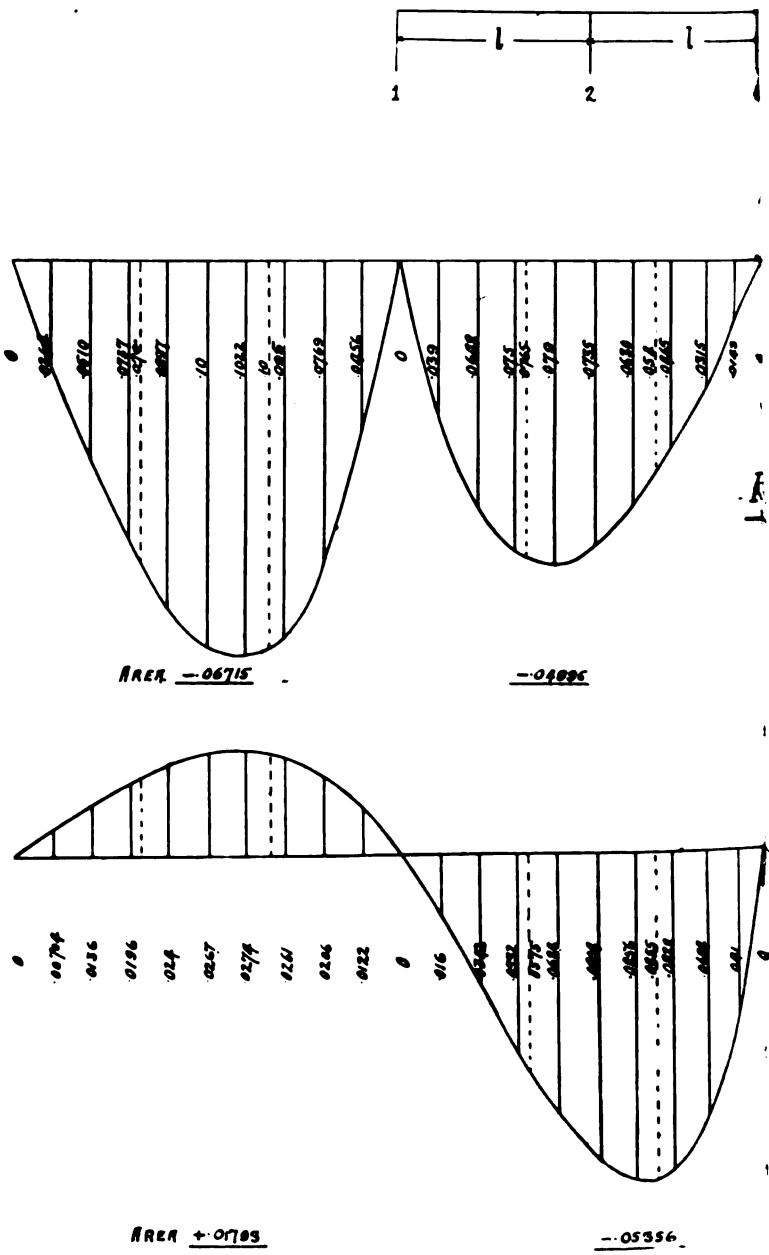

INFLUENCE LINES APPLIED

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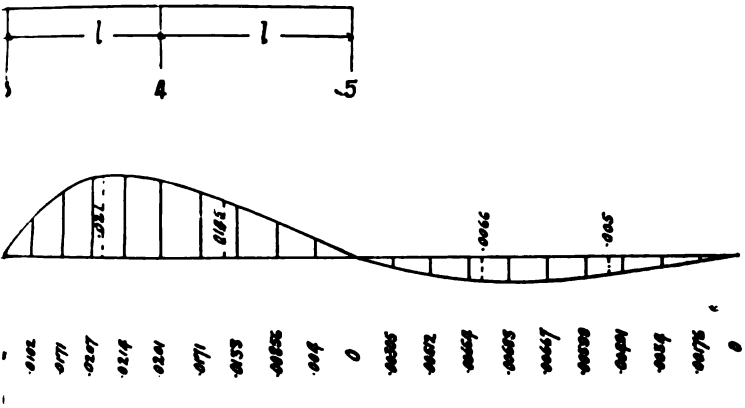
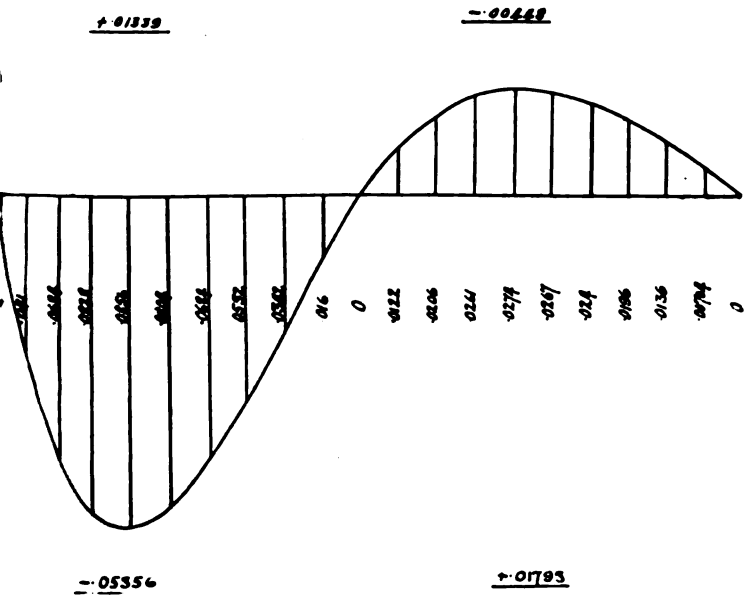


nfluence Lines for M_2 and M_5.

fig. V



TO CONTINUOUS BEAMS.

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From (A), (B) and (C) we obtain,

$$M_4 = \frac{1}{15}M_2; \quad M_3 = \frac{8}{15} - M_2$$

and $$M_2 = -0{\cdot}267P_1l(K - K^3).$$

Now let us take $P_1 = 1$ and $l = 1$. Then $M_2 = -0{\cdot}267(K - K^3)$ gives the values of the moment at support No. 2 caused by a load of unity acting at any point on the span 1–2 of unit length. The values of M_2, M_3 and M_4 for values of K from 0·1 to 0·9 are given in Table 1.

TABLE 1.

K	M_2	M_3	M_4
0·1	− ·0264	+ ·0070	− ·0018
0·2	− ·0510	+ ·0136	− ·0034
0·3	− ·0737	+ ·0196	− ·0049
0·4	− ·0897	+ ·0240	− ·0060
0·5	− ·1000	+ ·0267	− ·0067
0·6	− ·1022	+ ·0274	− ·0068
0·7	− ·0950	+ ·0261	− ·0065
0·8	− ·0769	+ ·0206	− ·0051
0·9	− ·0456	+ ·0122	− ·0030

Now let us suppose that the only load acting is P_2 on the span 2–3 and at a distance Kl from the left end of that span (*Fig.* 4), then

$$M_1 + 4M_2 + M_3 = -P_2l(2K - 3K^2 + K^3), \text{ but } M_1 = 0.$$

$$\therefore\ 4M_2 + M_3 = -P_2l(2K - 3K^2 + K^3).$$

Also $$M_2 + 4M_3 + M_4 = -P_2l(K - K^3)$$

and $$M_3 + 4M_4 = 0.$$

From these equations, putting $P = 1$ and $l = 1$, we can calculate M_2, M_3 and M_4 per unit load on span 2–3 only (*see* Table 2).

TABLE 2.

K	M_2	M_3	M_4
0·1	− ·0390	− ·0160	+ ·0040
0·2	− ·0648	− ·0342	+ ·0086
0·3	− ·0750	− ·0532	+ ·0130
0·4	− ·0790	− ·0672	+ ·0171
0·5	− ·0735	− ·0804	+ ·0201
0·6	− ·0630	− ·0856	+ ·0214
0·7	− ·0465	− ·0828	+ ·0207
0·8	− ·0315	− ·0684	+ ·0171
0·9	− ·0173	− ·0410	+ ·0103

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As the whole beam is symmetrical about support No. 3, we have now the necessary data for drawing influence lines for the moments at supports Nos. 2, 3 and 4 for any position of the concentrated load on any span (*see Fig.* 5). Ordinates drawn upwards from the horizontal base define positive moments and those drawn downwards negative moments. For dealing with uniform loads it is necessary to have the areas of the positive and negative loops of the curves marked on the diagrams.

As an example of the use of the influence line for four continuous spans, let there be a concentrated load of W lb. at the middle of each of the four spans (length of one span assumed to be 30 ft.). Then, since $K = 0{\cdot}5$ for each load, we have

$$M_2 = (-\cdot 1000 - \cdot 0735 + \cdot 0201 - \cdot 0067) \times W \times 30 \times 12 \text{ in.-lbs.}$$
$$= -\cdot 1601W \times 360 = -57{\cdot}8W \text{ in.-lbs.}$$
$$M_3 = (+\cdot 0267 - \cdot 0804 - \cdot 0804 + \cdot 0267)W \times 360$$
$$= -38{\cdot}6W \text{ in.-lbs.}$$

For uniform *dead* load of w_1 lbs. per foot run over all four spans, using the areas of the loops of the diagram,

$$M_2 = (-\cdot 06715 - \cdot 04896 + \cdot 01339 - \cdot 00448)w_1l^2$$
$$= -\cdot 1076w_1l^2, \text{ where } l = \text{length of a span.}$$

For uniform live load, w_2 lbs. per ft. run, we need only consider the areas of the loops corresponding to the loaded spans, e.g. live load on all four spans, live-load moment $M_2 = -\cdot 1076w_2l^2 = -\frac{1}{9{\cdot}3}w_2l^2$, just as for the dead load; but if there is no live load on span 3–4 then

$$\text{live load moment } M_2 = (-\cdot 06715 - \cdot 04896 - \cdot 00448)w_2l^2$$
$$= -\cdot 121w_2l^2 = -\frac{1}{8{\cdot}3}w_2l^2.$$

This is, of course, greater than $-\cdot 1076w_2l^2$, the live load moment when all spans are loaded.

Considering the influence line for the centre-support moment M_3, the following deductions are made. The maximum value of this moment due to uniform live load is obtained when the live load is only on the two spans nearest this support, and is

$$M_3 = -\cdot 1071wl^2 = -\frac{wl^2}{9{\cdot}34}.$$

If the live load is on all four spans

$$M_3 = (-2 \times \cdot 05356 + 2 \times \cdot 01793)wl^2$$
$$= -\cdot 07126wl^2 = -\frac{wl^2}{14}.$$

As the dead load must be on all spans simultaneously, the coefficient for the dead load moment must also be $-\frac{1}{14}$.

The bending-moment coefficients, $\frac{1}{12}$ for interior spans and $\frac{1}{10}$ for end spans,

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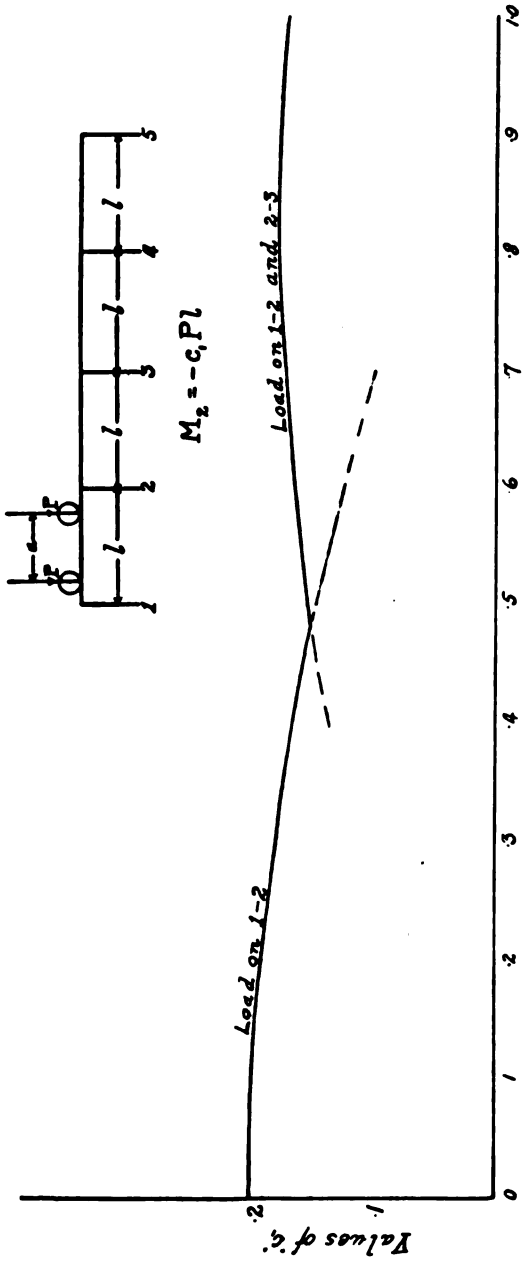

Maximum Values of M_2 for different Values of $\frac{a}{l}$.

fig IV

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which are incorporated in various building regulations, were imposed with the idea of securing safety in buildings where the superload might not be on all floors at once. In the design of structures under water or earth pressure in which all spans are necessarily live-loaded simultaneously the loading is less severe and the moments may be reduced. Examples of such structures are floors and walls of tanks, faces of counter-forted retaining walls, and foundation rafts. Influence lines for these give the proper coefficients at a glance, and their use results in more accurate and economical designs than are given by the usual moment coefficients.

By using influence lines the solution of a very usual case in practice, which is otherwise difficult, follows at once, viz., a continuous crane girder under the action of two equal rolling loads at a constant distance apart. It is only necessary to mark two points on a piece of paper to represent the axle centres to the same scale that the length unity of the influence line abscissa represents the span of the girder, then by shifting the line joining these two points along the horizontal base of the appropriate influence diagram until the sum of the ordinates under the two points representing the axle centres is a maximum the position of the crane carriage for a maximum moment at the support is readily found by trial, and the maximum value of the sum of the ordinates under the axle centres is the factor by which the product (length of span × *one* axle load) must be multiplied in order to obtain the moment at the support. In some cases the two wheels on one span will give the maximum moment at a support, but in others the wheels must be on opposite sides of the support. This difference is caused by the varying values which are possible for the ratio of the wheel-base to the length of a span.

The simple method just described will give sufficiently accurate results in practice even when there is a large number of unequally loaded axles, as in the case of a traction engine and trailers. By the application of the differential calculus to find the maximum values of M_2 for different values of the ratio

$$\frac{\text{distance between axle centres}}{\text{length of one span}} = \frac{a}{l}$$

Fig. 6 has been constructed. It will be observed that if $\frac{a}{l}$ is less than 0·48 the maximum value of M_2 is obtained with the crane on span 1–2, whereas if $\frac{a}{l} > 0{\cdot}48$ then the maximum value of M_2 occurs with one wheel on each side of support No. 2. The actual maximum moment M_2 can be calculated by multiplying the product Pl by the ordinate, over the given value of $\frac{a}{l}$, to the curve of *Fig.* 6.

Moment influence lines for the bending moments at the supports of continuous beams of (*a*) six equal spans, (*b*) four spans, the end ones being each of a length equal to 0·8 × length of an interior span, (*c*) six spans, the end ones being each of a length equal to 0·8 × length of an interior span, may be calculated by the same procedure as has been described. If the ends of the beam are considered as fixed we must assume two additional outside spans of infinitely short length. Owing to the rapid decrease in the lengths of the ordinates at a distance from the support at which we require the bending moment, we may neglect any spans

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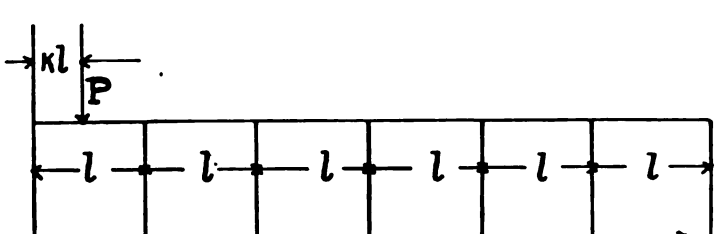

fig VII

which are beyond the third to the right or left of this support, or, in other words, we may consider any number of spans greater than six as being equivalent to six.

Tables 3 to 7 give the ordinates—for various arrangements of spans—required for drawing the moment influence diagrams for the supports. In all cases the supports are numbered from left to right, as in *Figs.* 4 and 7, No. 1 being at the extreme left end of the beam. K is also measured from left to right of each span. M_1, M_2, etc., denote the bending moment at support Nos. 1, 2, etc. The ordinate at each support is zero. The spans between supports Nos. 1 and 2 and between Nos. 2 and 3 are denoted by "Span 1–2" and "Span 2–3" respectively.

TABLE 3.

Four equal spans, ends simply supported.
Influence Line for M_3.

Ordinates.

K	Span 1–2	Span 2–3	Span 3–4	Span 4–5
0	0	0	0	0
·1	+ ·0070	− ·0160	− ·0410	+ ·0122
·2	·0136	·0342	·0684	·0206
·3	·0196	·0532	·0828	·0261
·4	·0240	·0684	·0856	·0274
·5	·0267	·0804	·0804	·0267
·6	·0274	·0856	·0684	·0240
·7	·0261	·0828	·0532	·0196
·8	·0206	·0684	·0342	·0136
·9	·0122	·0410	·0160	·0070
1·0	0	0	0	0
Area of loop	+ ·0179	− ·0536	− ·0536	+ ·0179

The influence line for M_3 (Table 3) has been plotted in the lower half of *Fig.* 5. It is convenient to draw dotted ordinates at the third points of the spans, *i.e.* for $K = 0{\cdot}33$ and $K = 0{\cdot}67$, and write their lengths on them.

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TABLE 4.

Four spans, ends simply supported, exterior spans of a length equal to 0·8 × length of interior spans.
Influence Line for M_3.

Ordinates.

K	Span 1–2	Span 2–3	Span 3–4	Span 4–5
0	0	0	0	0
·1	+ ·0050	− ·0148	− ·0414	+ ·0087
·2	·0098	·0321	·0680	·0141
·3	·0139	·0496	·0812	·0181
·4	·0171	·0662	·0840	·0195
·5	·0190	·0780	·0780	·0190
·6	·0195	·0840	·0662	·0171
·7	·0181	·0812	·0496	·0139
·8	·0141	·0680	·0321	·0098
·9	·0087	·0414	·0148	·0050
1·0	0	0	0	0
Area of loop	+ ·0101	− ·0516	− ·0516	+ ·0101

TABLE 5.

Same beam arrangement as in Table 4.
Influence Line for M_2.

Ordinates.

K	Span 1–2	Span 2–3	Span 3–4	Span 4–5
0	0	0	0	0
·1	− ·0187	− ·0490	+ ·0115	− ·0024
·2	·0363	·0801	·0189	·0039
·3	·0518	·0880	·0226	·0050
·4	·0637	·0890	·0234	·0054
·5	·0709	·0850	·0217	·0053
·6	·0726	·0710	·0184	·0048
·7	·0675	·0540	·0138	·0039
·8	·0528	·0350	·0089	·0027
·9	·0325	·0170	·0041	·0014
1·0	0	0	0	0
Area of loop	− ·0378	− ·0574	+ ·0145	− ·0028

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TABLE 6.

Six spans, ends simply supported, end spans of a length equal to 0·8 l, where l = length of each of the four interior spans.
Influence Line for M_4 (middle support).

Ordinates.

K	Span 1–2	Span 2–3	Span 3–4	Span 4–5	Span 5–6	Span 6–7
0	0	0	0	0	0	0
·1	− ·0014	+ ·0040	− ·0153	− ·0418	+ ·0112	− ·0024
·2	·0027	·0087	·0329	·0696	·0182	·0039
·3	·0038	·0136	·0514	·0818	·0218	·0049
·4	·0047	·0179	·0672	·0846	·0225	·0053
·5	·0052	·0211	·0790	·0790	·0211	·0052
·6	·0053	·0225	·0846	·0672	·0179	·0047
·7	·0049	·0218	·0818	·0514	·0136	·0038
·8	·0039	·0182	·0696	·0329	·0087	·0027
·9	·0024	·0112	·0418	·0153	·0040	·0014
1·0	0	0	0	0	0	0
Area of loop	− ·0028	+ ·0134	− ·053	− ·053	+ ·0134	− ·0028

It will be noticed that the curve is symmetrical about a vertical line through support No. 4.

TABLE 7.

Same arrangement of beams as for Table 6.
Influence Line for M_2.

Ordinates.

K	Span 1–2	Span 2–3	Span 3–4	Span 4–5	Span 5–6	Span 6–7
0	0	0	0	0	0	With load on this span, the ordinates are negligible.
·1	− ·0191	− ·0434	+ ·0117	− ·0031	+ ·0008	
·2	·0368	·0712	·0189	·0052	·0014	
·3	·0527	·0848	·0226	·0061	·0016	
·4	·0647	·0880	·0237	·0063	·0017	
·5	·0720	·0822	·0226	·0059	·0016	
·6	·0738	·0700	·0192	·0051	·0013	
·7	·0690	·0531	·0145	·0039	·0010	
·8	·0554	·0345	·0078	·0023	·0007	
·9	·0329	·0144	·0042	·0011	·0003	
1·0	0	0	0	0	0	
Area of loop	− ·0385	− ·0546	+ ·0147	− ·0039	+ ·0011	0

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N.B.—The moments are given in terms of the longer (interior) spans. *E.g.* for uniform load on all spans, if l = length of an interior span,

$$M_2 = (-\cdot 0385 - \cdot 0546 + \cdot 0147 - \cdot 0039 + \cdot 0011)wl^2$$
$$= -\cdot 0812wl^2 = -\frac{W}{12} \textit{ quam proxime},$$

as we already know is the case for such an arrangement of spans.

When the influence lines have been plotted and the areas of the loops ascertained it is advisable to write down the sum of the areas of the negative loops and the sum of the positive areas for reference when dealing with uniform loads.

It is a simple matter to calculate the ordinates to the influence lines for moments at *any* point, *e.g.* the centre, of *any* span when we have the moments at the supports, but it must not be forgotten that the moment at the *centre* of a span of a continuous beam is not necessarily the maximum positive moment on that span.

The Automatic Sealing of Leaks through Concrete.

For a long time it has been known that the watertightness of concrete increases with its age; when reservoirs and pipes are first put into use white deposits, composed of carbonate of lime, appear on the sides opposite to the water. The Zurich Water Company has, states *Le Constructeur de Ciment Armé*, carried out a series of tests to determine the degree of impermeability of concrete and the increase in the impermeability with time. The first specimen to be tested was 30·7 in. in diameter and 8·67 in. thick. The quantities of material used in the mixing of the concrete were, per cubic yard of finished concrete, 670 lbs. of cement, 33½ gallons of water, and 39 cu. ft. of aggregate composed of 7 parts of coarse aggregate (¼ in. to 1½ in.) to 5 parts of fine aggregate (under ¼ in. gauge).

The block was made on November 9 and tested from December 12 to March 10 of the following year under an average pressure of 74 lbs. per sq. in., with the following results:

Date.	Average Water Percolating per day.
December 15 . . .	71·00 cu. in.
,, 15 to 30 .	13·50 ,,
,, 30 and after	1·07 ,,
January 29 . . .	0·12 ,,

(The last figure was due to leakage through the joints of the apparatus.)—W. S. G.

Littleton Reservoir.

Occupying an area of nearly 800 acres, the new reservoir at Littleton, Staines, is to be officially opened by H.M. the King on June 13. The construction of this reservoir has taken about ten years, including an interval of five years when work was brought to a standstill by the war.

The area of the reservoir is surrounded with a concrete embankment rising 38 ft. above ground level, battered on the inside to a gradient of one in three. The maximum depth of water when it is full will be 38 ft. A breakwater extends from one side to a little over halfway across the basin for the purpose of breaking the waves caused by high winds and also to screen the outlet; this baffle is six furlongs in length. Sir James Restler, late Chief Engineer to the Metropolitan Water Board, was chiefly responsible for the design, and the work has been completed under the direction of Mr. Henry E. Stilgoe, the present Engineer to the Board.

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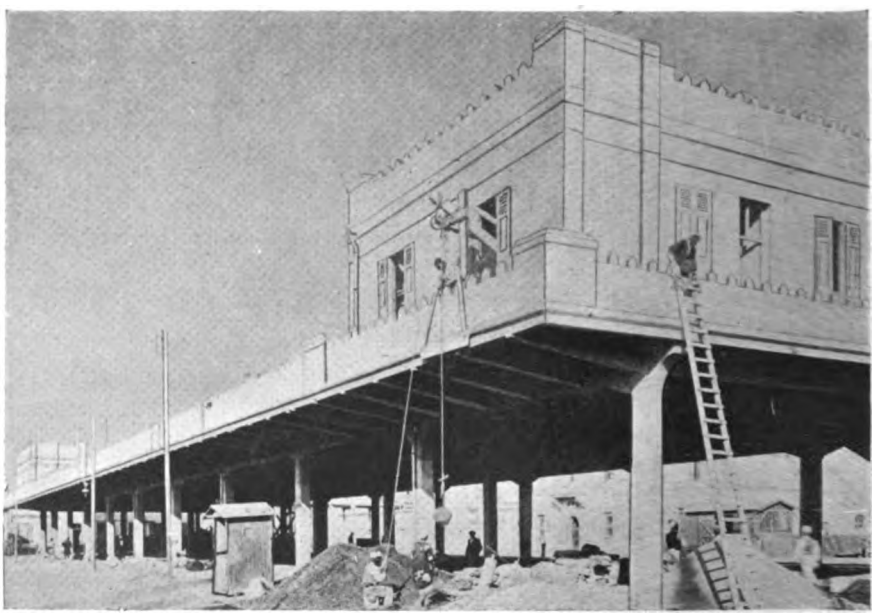
Reinforced Concrete Roof in Tunisia.

By P. Calfas, Ing.E.C.P.

On the north-west quay of the Harbour of Sfax, Tunisia, a reinforced concrete roof has recently been built in order to protect casks containing olive oil from the sun. The roof measures 74 ft. 5 in. in width by 313 ft. 6 in. in length, and is supported by 33 pillars spaced at intervals of 27 ft. across and 30 ft. 8 in. along the length of the structure. At each end small buildings have been erected on the roof. The work is constructed on a very compressible soil, which has been made up with materials obtained from the dredging of the harbour. The soil is so unstable that the foundations of an hotel in the vicinity were found to have sunk by 11¾ in., and in order to avoid a similar occurrence each pillar supporting the roof to the new storage place rests upon a square raft of large area and the framing consists of cantilever beams upon which rest intermediate secondary and jointed beams. Thus if the settling of the foundation is not regular the cantilever beams and the intermediate beams will assume an inclination relative to each other and cracks will be avoided.

Precautions have also been taken to raise the columns if necessary. With this end in view the pillars have not been embedded in the raft but rest on a spherical ball-joint. The pillars are provided near their base with an aperture, or slot, allowing for the introduction of a cross-beam or bar. For lifting up any one of the pillars, the cross-beam is introduced in the slot and raised up by means of two jacks bearing directly on the foundation raft. The pillars may thus be brought up to their former level, and supported by introducing aluminous cement into the gap at the ball-joint. By means of this device the roof can always be brought up to its former level, if any of the columns sink before the final settling of the base foundations has been reached. The contractors are Messrs. Vidrecourt and Cadré, and the engineer M. H. Lossier.



Reinforced Concrete Roof in Tunisia.

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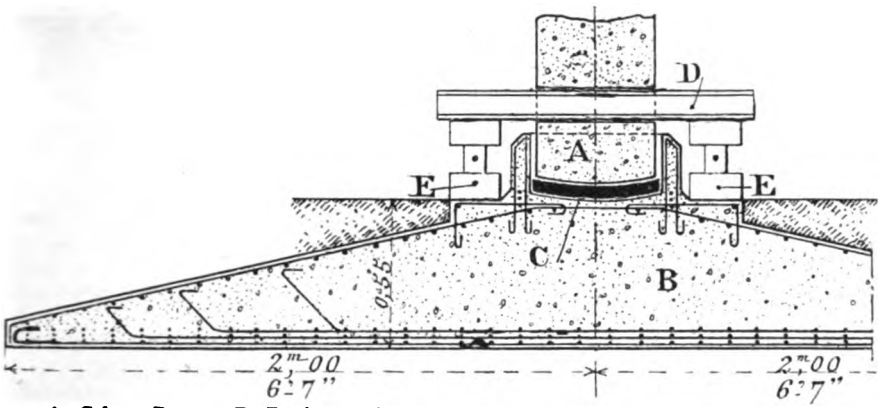

A—Column Base. B—Footing. C—Joint. D—Cross Beam. E—Lifting Jacks.

Reinforced Concrete Roof in Tunisia: Section showing Foundation of Columns.

Book Reviews.

Structural Engineering (Strength of Materials) By George Fillmore Swain, LL.D.
London: McGraw-Hill Book Co. 569 pp., 468 illustrations. Price 25s. net.

THIS volume is the first of a series dealing with the theory and design of structures. The intention of the author is to publish four volumes in all, under the heading of "Structural Engineering," the second volume being devoted to fundamental properties of materials, while the third and fourth will complete the theory and design of structures, simple and statically indeterminate respectively. It has been the aim of the author to present his subject in as complete a form as possible, and the book under review, which deals with the strength of materials, certainly fulfils his object, and goes into detail without, however, obscuring the principle.

The book gives a very complete and clear discussion on the fundamental principles of the strength of materials applicable in the design of structures of various kinds, and also contains all that is needed for the student of machine design; the illustrations are excellent, and the text contains so many references that it may well serve as a guide for those who wish to go in for a more extensive study and pursue the subject as far as our present knowledge permits. On the other hand, it is realised that this volume may go beyond the limits of the course provided by some engineering colleges, and the author has therefore made a suggestion for an abridged course and gives a list of parts that may be omitted, subject to the discretion of the teacher. Professor Swain's method of presenting the various principles is in many cases refreshingly novel and original, and the last chapter on "Mathematics, Testing, and Common Sense," which gives the author's own view point, reveals the Professor as an eminently practical and experienced engineer himself.

The book is equally handy for the university student and the engineer, and should be in the possession of everyone who appreciates a really good book on this branch of technical literature.

R.N.S.

Theory and Design of Structures. By Ewart S. Andrews.
London: Chapman & Hall, Ltd. Price 13s. 6d. net.

THIS is the fourth edition of Mr. Andrews' book, and differs from the previous editions chiefly in the treatment of columns. As the previous editions have already been reviewed it is unnecessary to review the present one in detail, and we would confine ourselves to stating that it is a work which can be thoroughly recommended.

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Tables for Reinforced Concrete Floors and Roofs. By R. Travers Morgan.

London: Chapman & Hall, Ltd. Price 10s. 6d. net.

THIS book consists of tables giving the thickness of slabs and the steel reinforcement required for inclusive loads varying from 50 lbs. to 400 lbs. per sq. ft. with stresses of 600 lbs. in the concrete and 16,000 lbs. in the steel, the bending moment being taken at $\frac{wl}{8}$, $\frac{wl}{10}$ or $\frac{wl}{12}$.

Handbuch für Eisenbetonbau. By Dr. F. Emperger.

Third Edition, Volume 13. Buildings for Special Purposes (Industrial Buildings). 706 pp. 1127 illustrations. W. Ernst & Sohn, Berlin: Price 46.20 frcs. (Swiss).

THIS volume is the last but one of the fourteen which comprise the well-known handbook on reinforced concrete by Dr. Emperger. Together with volume 14, which forms a continuation and deals with silos and agricultural buildings, this volume treats the application of reinforced concrete for industrial buildings of all descriptions and is a fair example of the usual thoroughness and clear style which is characteristic of the works by this author. The book contains a wealth of information and descriptive illustrations, and goes into details to a degree that is extremely useful for any designer who is interested in the subject. Amongst the various kinds of buildings dealt with in the five chapters may be mentioned offices and business houses, market halls, abattoirs, cold storage buildings, churches, theatres and public halls, factories, warehouses, chimneys, etc. Each chapter has an introductory review of its subject, after which it treats the technical and economical aspect of the matter in all its bearings, and concludes by giving an abundance of examples of buildings actually executed. There are many interesting solutions of difficult statical problems mentioned in the book, and a chapter which should appeal to the up-to-date designer is the one dealing with cold storage buildings, an example of industrial structures which are beginning to claim the attention of the engineer in this country, more particularly after the lessons learnt during the war. The last chapter, on chimneys, deals very exhaustively with the subject, the analytical treatment of the calculations in particular being very fully elucidated.—R.N.S.

The Empire Municipal Directory and Year Book.

London: *Municipal Engineering*. Price 11s.

THIS valuable Directory and Year Book is now in its 43rd year. Not only is it claimed that it constitutes a complete encyclopædia on all aspects of municipal work, but that it is the only publication which contains complete and officially-corrected lists of all the corporations, county, urban and rural district councils in Great Britain and Ireland, and our Overseas Dominions, and their chief officials, viz., surveyors, water, gas, electric lighting, and tramway engineers, clerks, medical officers of health, sanitary inspectors, and others. The Foreword is by Sir Henry Maybury, Director-General of Roads. The special articles constitute a review of progress in the past year, and a record of present practice on the following subjects: Roads and road-making; street lighting; motor vehicles for municipal services; water purification and softening; swimming baths; sewerage and sewage disposal; etc., together with a diary, three days to a page. The volume is invaluable to all who have business relations with local authorities, or who are interested in local government work.

Design of Formwork.

OWING to pressure of other matter, Part VIII of Design of Formwork for Reinforced Concrete Construction, by A. E. Wynn, is unavoidably held over this month. We hope, however, to continue the series in our next issue.

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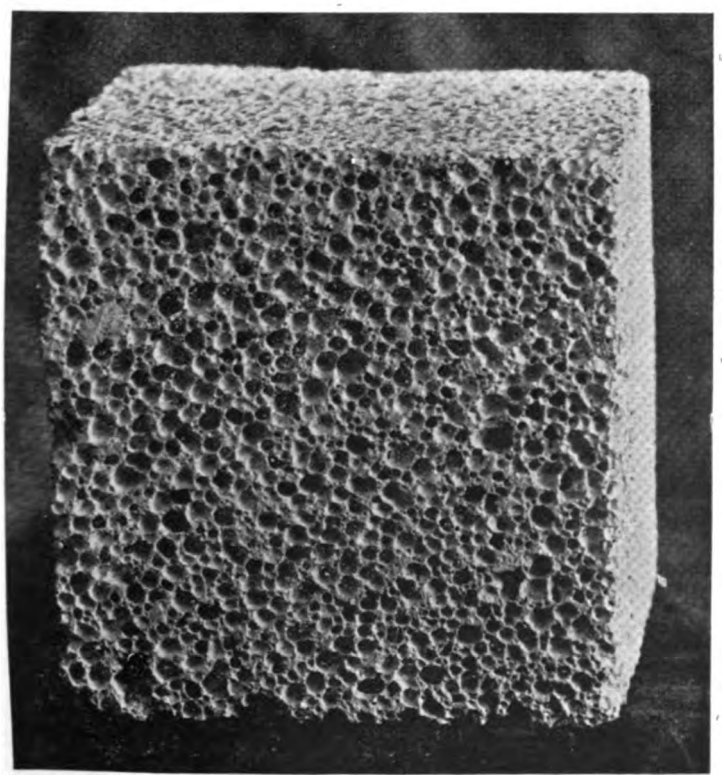
Expanded Concrete.

By K. P. Billner.

MUCH scientific thought has been spent in efforts to ascertain how a mixture of cement, aggregates, and water should be proportioned in order to obtain the most dense and consequently the strongest concrete. A less desirable result of the density of concrete is that dense concrete is a good conductor of heat and cold; in other words, it is a less desirable material when it is to be used as protection against heat or cold. A remarkable change in this respect has been accomplished by the invention of expanded concrete. The only country where expanded concrete is being manufactured to any extent is Sweden, where the material is now being manufactured by four different firms.

Expanded concrete can be made either by a chemical process or mechanically. The chemical method has given the best results, and is therefore employed. This method is based upon a reaction between an alkaline metal and water when mixed with cement and other ingredients. The water is dissolved into its elements, the oxygen makes an alliance with the metal, and the free hydrogen creates little bubbles in the mixture, which swells up to several times its original



EXPANDED CONCRETE: ACTUAL SIZE.

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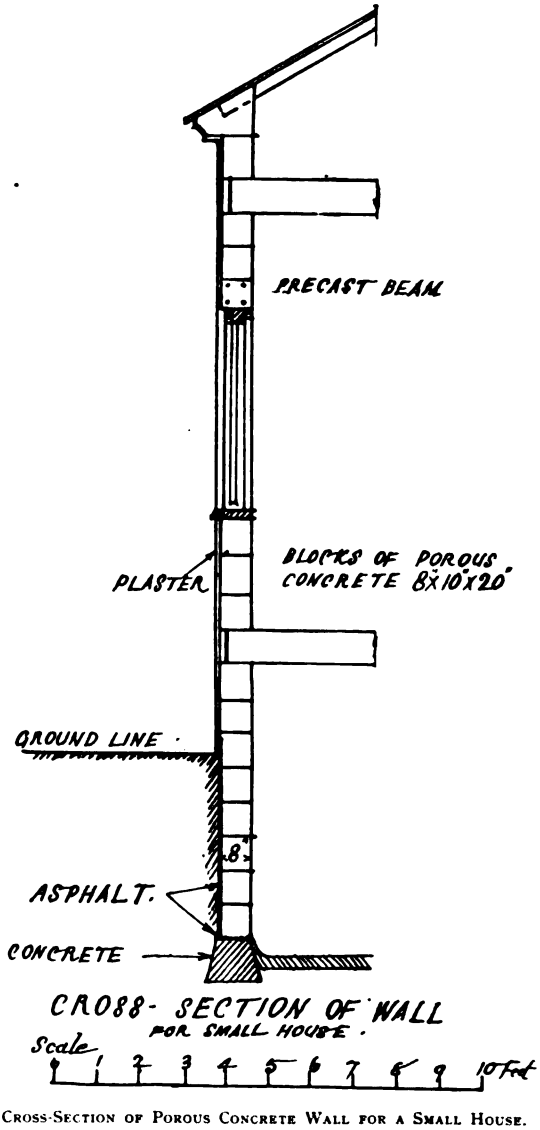
volume and then gradually hardens in the same way as ordinary concrete. It is possible in this manner to produce a concrete so light that a cubic foot thereof weighs only 20 lbs. (compared with 130 lbs. per cubic foot for ordinary concrete).



Cross-Section of Porous Concrete Wall for a Small House.

The strength, however, decreases rapidly when the material is made very light. For building purposes a material weighing about 42 lbs. per cub. ft. is most suitable. At this weight the ultimate compressive strength averages 400 to 500 lbs. per sq. in. This allows for a safety factor of about 10 when the material is used for walls in two-story buildings.

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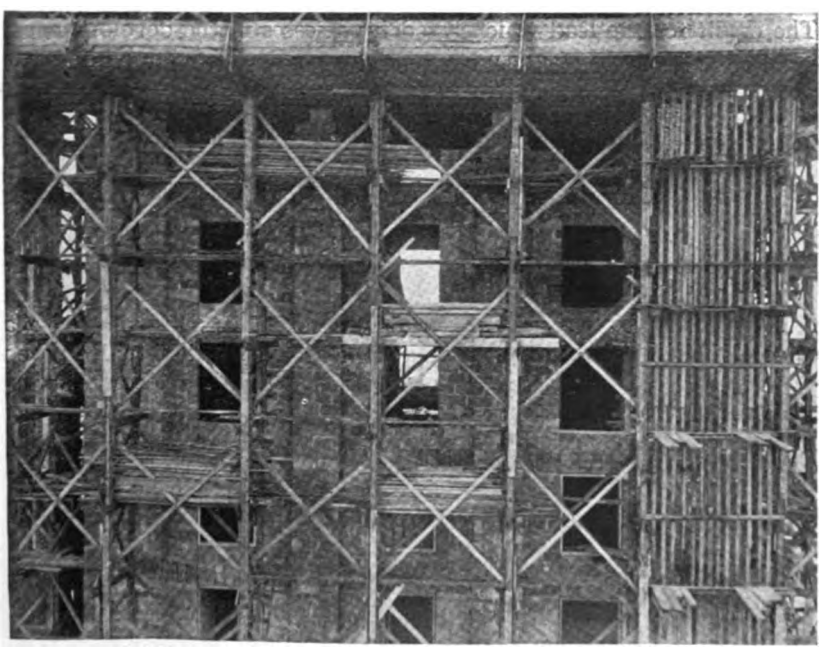
The texture of the material resembles that of cork. It is full of small and evenly distributed cavities, which make up 70 per cent. of its volume. Each one of these cavities is a closed compartment surrounded by thin walls, and this is the reason the material has such exceptionally good insulating qualities. Thorough tests have been made under the auspices of the Swedish Academy of Engineering Science in order to ascertain the amount of heat transmission through walls of this material in various thicknesses; the accompanying chart shows coefficients of heat transmission through expanded concrete, brick, mortar and ordinary concrete, and also thicknesses of these materials required to give the same amount of protection. It will be seen that the heat insulating properties of expanded concrete are more than three times greater than those of brick, and seven times greater than those of ordinary concrete.

The material can be made either in blocks or it can be cast *in situ*. If the latter method is employed the following advantages accrue: (1) Unskilled labour may be used, (2) there will be no mortar joints in the walls, and (3) the walls may be reinforced. The author has made a series of tests in order to ascertain the amount of adhesion between expanded concrete and steel reinforcement, and he has found the bond to be proportionate to that which takes place between steel and solid concrete. It is therefore possible to make beams for spanning over door and window openings instead of arching them. The material can also be used for floor construction, and such floors are considerably lighter than ordinary reinforced concrete floors.

Another advantage of expanded concrete is that it can easily be worked with



View of a 16-story Building in Stockholm: Reinforced Concrete Frame with Outside Walls between Columns of Expanded Concrete Blocks 8 in. thick.

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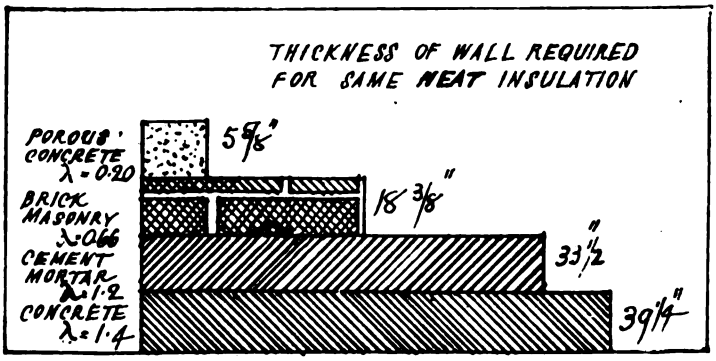

common wood-working tools. It can be sawed, and grooves can be made in it for electric and other conduits. In this respect, and also in respect to protection against heat or cold, it is greatly superior to hollow concrete tiles.

Besides the quality of good heat insulation, the lightness of expanded concrete is one of its most valuable qualities. A building of this material does not weigh more than half as much as a brick building of the same proportions. This is of importance when building on a poor foundation; it is also of importance when used in modern business buildings with steel or reinforced concrete frame construction. Here it will open possibilities for saving of material both in the supporting frame and in the foundations. With regard to cost, expanded concrete can be made at about the same price as ordinary concrete of equal volume.

The credit for the lead which Sweden possesses in the development of this building material should be given to the architect Axel Ericson, who first worked out a process for its manufacture. Among the firms which now produce the material are two cement factories, and the writer has organised the plant and methods employed by one of these, i.e. the "A. B. Vallevikens Cementfabrik."

Finally, it should be borne in mind that expanded concrete is practically the same substance which composes ordinary concrete, the only building material which, when rightly made, increases in strength as time goes on.

A house built of expanded concrete has been built in the Housing Section of the Palace of Engineering at the British Empire Exhibition, and the material has been tested by Messrs. Holland & Hannen and Cubitts, Ltd., in whose laboratory pieces of expanded concrete have been floating in water for over five weeks without any signs of absorption. Samples of the material have also been brought to red heat and then immediately immersed in water three times in succession, the only result being a crumbling of the surface.

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Modern Methods of Construction.

In our issue for April we gave the first part of a paper read by Dr. Oscar Faber, O.B.E., D.Sc., M.Inst.C.E., before the Royal Institute of British Architects. In the second part of the paper the practical side of foundation work was dealt with, as follows :—

The design and construction of foundations has been subject to gradual growth and development as fresh problems presented themselves with the taller and heavier structures which are now demanded, and also as a result of comparatively modern materials such as steel and reinforced concrete. Some of the examples given may be familiar ; it is hoped, nevertheless, that a description of actual difficulties experienced in practice and the methods by which they were solved may be suggestive and serve a useful purpose.

Ferro-concrete Rafts.

The author has been much impressed with the virtues of ferro-concrete rafts in many situations for solving foundation problems. As ferro-concrete itself is for practical purposes only from 20 to 30 years old, such construction may legitimately be considered to be modern.

Among the principal advantages of concrete rafts as foundations may be included the following :

(1) The pressure is distributed over the maximum possible area of soil, and therefore the pressure per sq. ft. is reduced to a minimum.

(2) Any tendency to unequal settlement is minimised, as a raft properly designed has a certain power of bridging over soft places and applying the loads to those parts best able to receive them.

(3) The raft also forms a tie preventing the lateral spreading of foundations, which is sometimes important.

(4) In the case of buildings with deep basements, where the lower floor is frequently far below water level, some sort of raft construction is necessary to receive the upward water pressure and prevent it from flooding the basements. In these cases the raft and the retaining walls form together a box which, by means of asphalte or other means, is rendered watertight.

Reviewing these points in detail, it frequently happens nowadays that a soil which would be quite insufficient with the normal pressures which isolated foundations would produce on the soil can be founded upon quite safely under a well-designed raft, and so avoid the necessity for sinking to great depths to secure a foundation of sufficient carrying capacity. The saving will, of course, depend on the depth at which this better foundation is found, and is enhanced when special difficulties, such as water and perhaps the necessity of underpinning surrounding buildings, are entailed in this process of digging to great depths. It frequently happens that a concrete raft can also be used as a floor in the building, and so increased economy is obtained. It sometimes requires a good deal of courage to found on soils which in the ordinary way would be considered quite dangerous, and yet the author's experience is so favourable with these concrete rafts when properly designed as to give him a great deal of confidence.

I would like to refer to one or two special examples of small rafts which, in spite of their being small, have special points of interest. One of these is represented by the rafts for the laundry and power house at Marlborough College, for which Messrs. W. G. & H. A. Newton were architects. (See *Figs.* 2, 3, 4 and 5.)

This is founded on very soft low-lying soil, and it was desired to keep the ground floor 18 in. above the general level of the ground owing to the risk of floods. A raft foundation was used, the portion near the walls being about 9 in. thick and the top level with ground level. The portion between the walls is raised up above the ground so as to form the ground floor and actually rests on hardcore rolled on to the mud to receive it. In this way the raft also forms the ground floor, and so makes a considerable economy, and deep foundations below water-logged ground were entirely avoided. There has been no settlement, in spite of the fact that heavy engines supplying both the College and the town with electric light are founded on the raft and produce considerable vibration.

(*To be continued.*)

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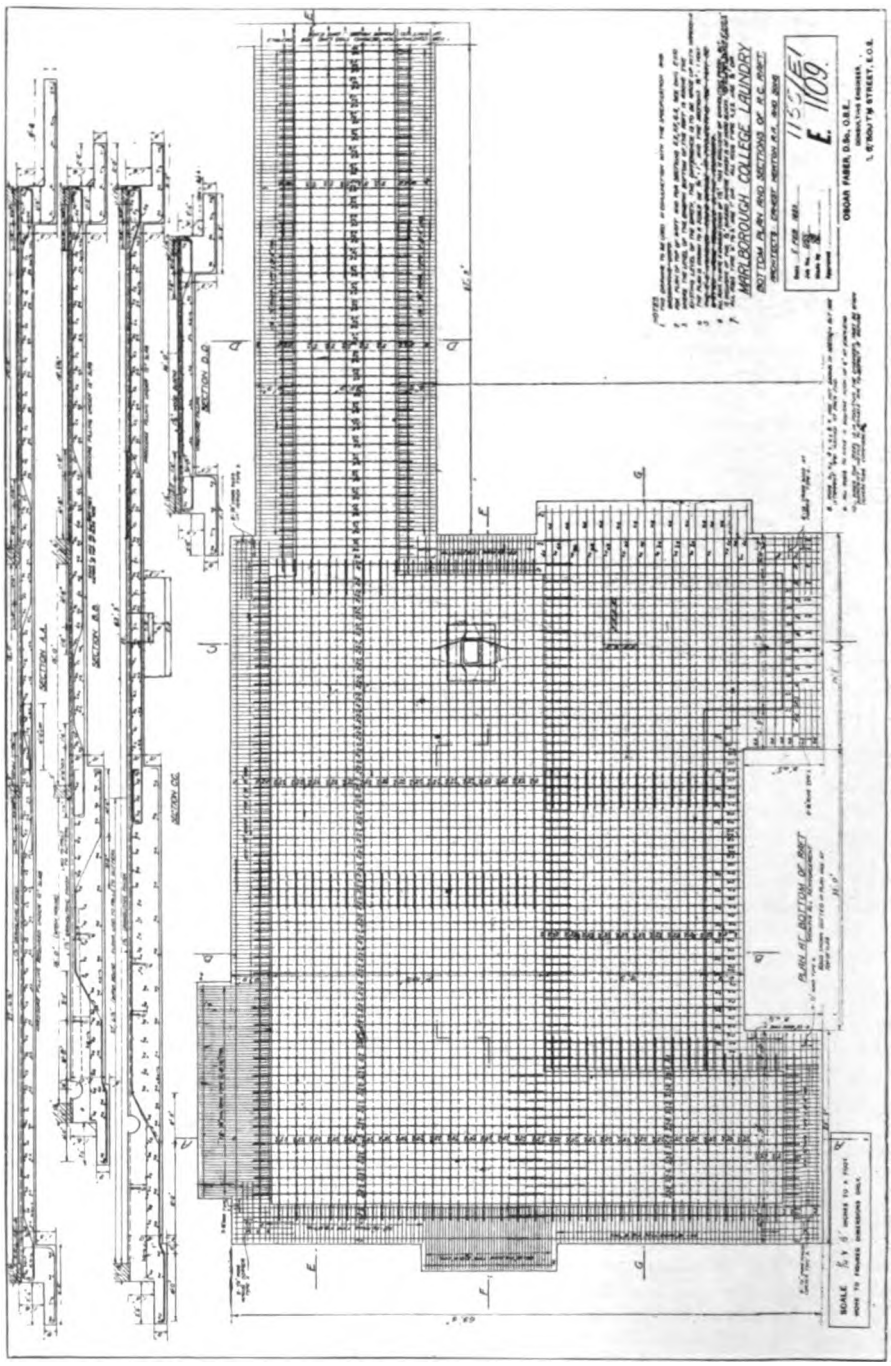

Fig. 2.—Reinforced Concrete Raft at Marlborough College. (*See p.* 329.)

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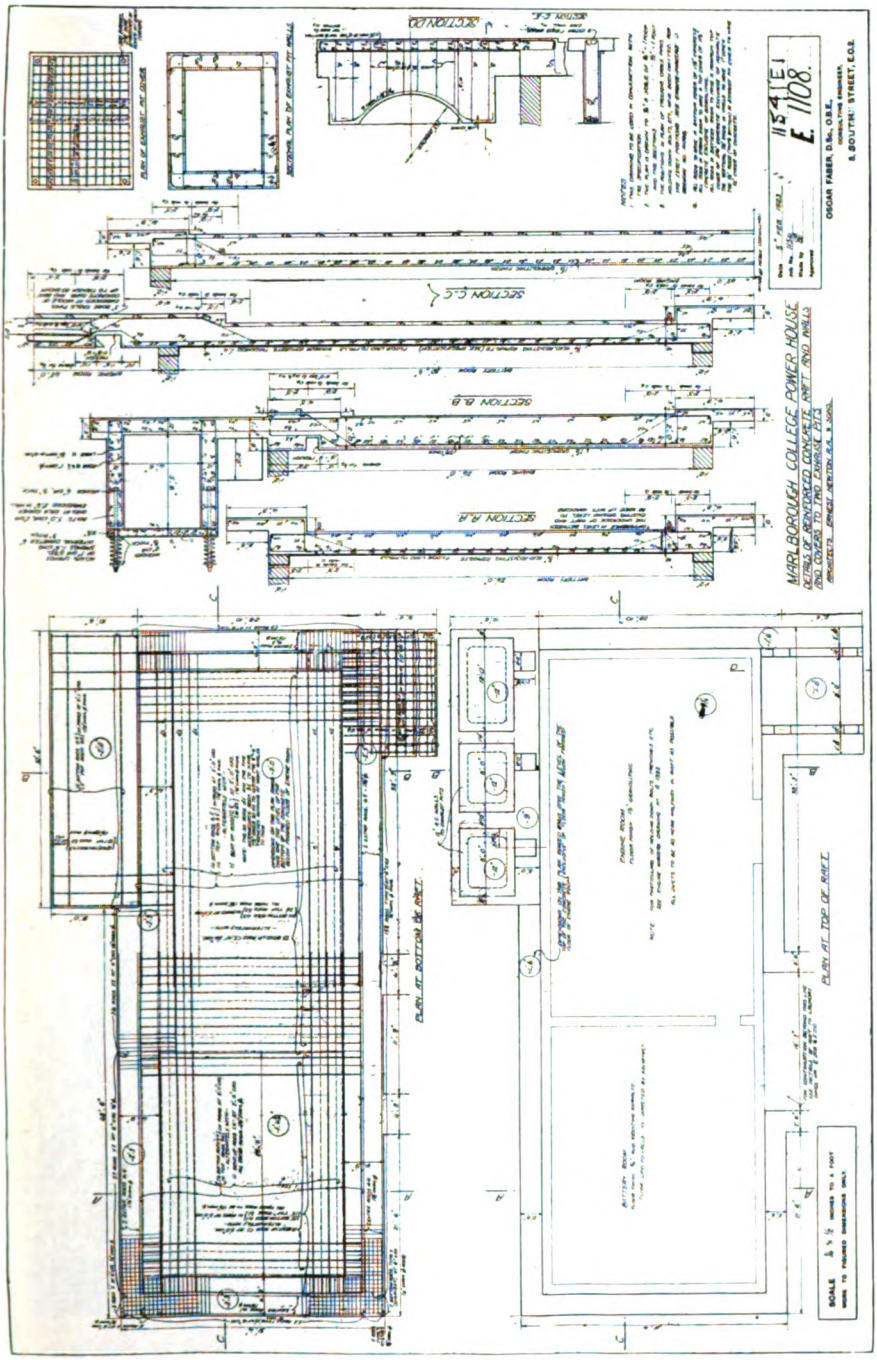

Fig. 3.—Reinforced Concrete Raft at Marlborough College. (*See p.* 329.)

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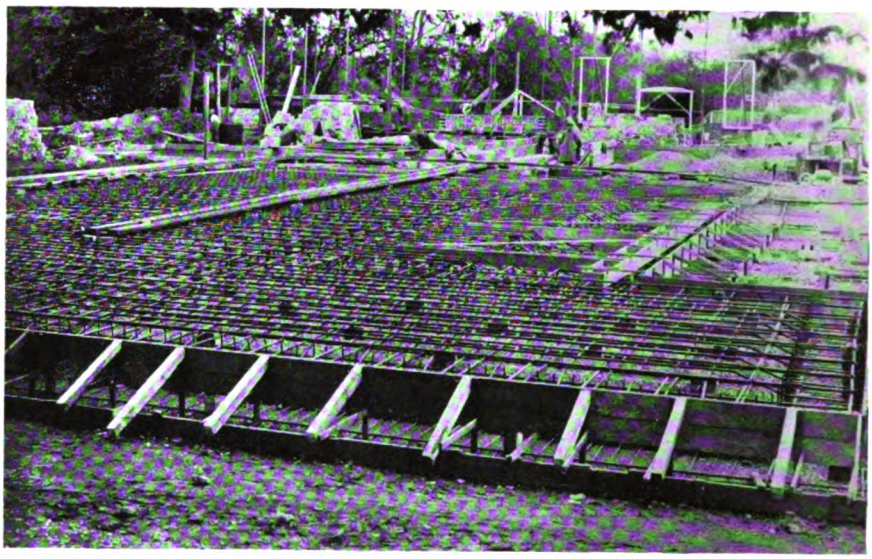

Fig. 4.

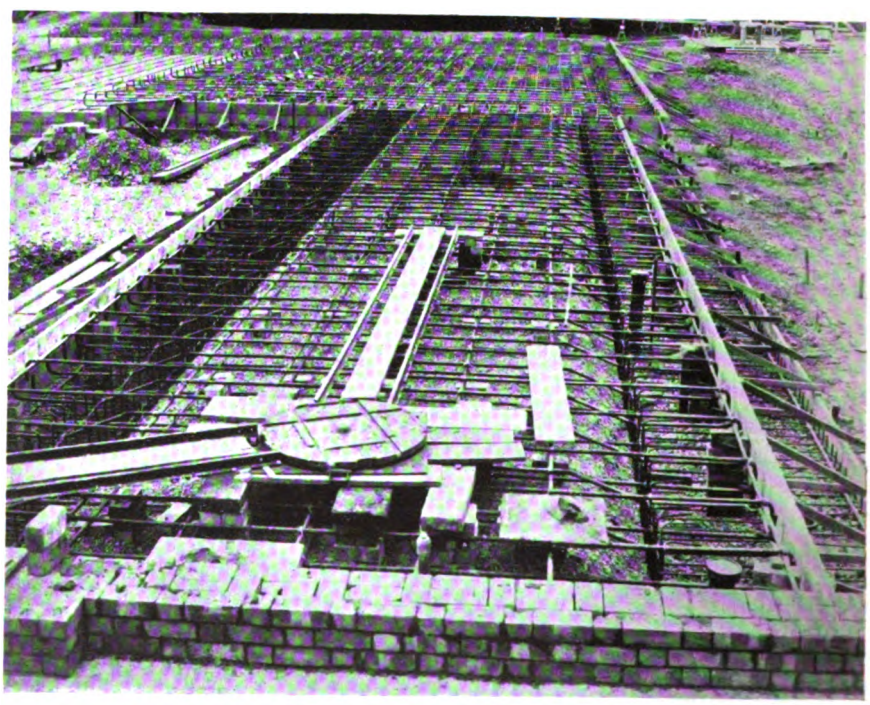

Fig. 5.

Reinforced Concrete Wharf at Marlborough College. (*See p.* 329.)

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Concrete Practice in the United States and London.

Report by the Architect to the London County Council.

Mr. G. Topham Forrest, F.R.I.B.A., Architect to the London County Council, who spent two months in the United States last autumn studying the methods of construction and building regulations in that country, has now issued a voluminous report on his investigations, and makes a number of recommendations and suggestions as to how building practice in this country, the London County Council Regulations, and the various Building Acts might be brought up to date and revised to meet modern conditions. Town-planning, educational buildings, art commissions, traffic control, theatres, railway stations, by-laws, fire escape, concrete and steel frame construction, are among the subjects dealt with at length and illustrated by a number of diagrams, plans, and drawings.

On the question of building regulations generally, it is pointed out that in the New York administration there is greater flexibility than in London owing to the fact that in New York the building laws are subject to periodical revision, and it is recommended that in any revision of the London Building Acts all matters that can be so dealt with should be removed from the Acts and incorporated in by-laws subject to revision by the Council. It is considered that generally the London regulations compare favourably with those of New York and other cities in the United States.

In dealing with foundations, New York Building Code provides a presumptive scale for the sustaining power of the soil from 1 ton per sq. ft. for soft clay up to 40 tons per sq. ft. for hard rock, and the footings are arranged so that such capacities are not exceeded. The Superintendent may require the sustaining power of the soil to be tested. Foundations are required to be at least 4 ft. below ground level, and any metal therein must be protected from rust. The rules for foundations apply to all buildings. Rules are provided for the use and driving of piles. Retaining walls are required to be calculated to resist all pressures on them. In the London Building Act, 1894, so far as ordinary foundations in London are concerned, there is only one definite requirement as regards concrete, and that is contained in the by-laws made under the Metropolis Management and Building Acts Amendment Act, 1878, where it is required that the foundations of the walls of every house or building shall be formed of a bed of good concrete not less than 9 in. thick and projecting at least 4 in. on each side of the lowest course of footing of such walls. The by-laws also authorise District Surveyors to permit the omission of concrete altogether where the footings rest upon a natural bed of gravel. A modern building of 80 ft. in height with basements in addition and two stories in the roof can therefore be constructed upon gravel soil without any concrete under the footings if the District Surveyor will so permit. By the London County Council (General Powers) Act, 1909, which deals with steel-frame and reinforced concrete buildings, the foundations are required to be so arranged as to come within a definite schedule of soil pressure. It would appear, therefore, that if any building is constructed under the 1894 Act with concrete 9 in. thick under the footings and the builder declines to provide any thicker concrete, the District Surveyor (unless such building were a public building coming under Section 78 of the 1894 Act) could only take action when the walls had in his opinion become a dangerous structure.

Mr. Topham Forrest is therefore of opinion that the provisions of the 1909 Act should be extended in principle to all buildings the construction of which comes within the ordinary rules of the 1894 Act, and recommends that the dimensions of foundations of walls and permissible maximum pressures should be prescribed in the Building Acts. It is also recommended that the schedule of wall thicknesses should be reviewed and amended as found desirable with a view to fixing maximum stresses on brickwork.

On the subject of fire-resisting construction, the opinion is expressed that the provisions of the London Building

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Acts are adequate for the class of buildings usually erected in London.

In discussing concrete and reinforced concrete, the author of the report states that his visit has confirmed his opinion that the regulations should be revised at frequent intervals in order to keep pace with developments, and the investigations may be summarised as follows :—

CORROSION OF STEEL.—The only concretes which will ensure the continued brightness of embedded steel are those which are made of inert materials, and which, to-day, are occasionally regarded as containing some excess of cement. While the use of clinker and coke breeze aggregate may tend to reduce cost, it may reduce the life of the structure by its corrosive action on the reinforcement. Concretes of greater strength and density are impervious, and will protect any embedded steel. It would be better to specify the volume of aggregate including sand instead of specifying so much gravel plus sand; this would allow greater freedom to the designer, and as good results obtained.

Recommendation.—That in the case of concrete specifications, wherever tests are made to ensure that there is sufficient sand to fill voids in the gravel, the specifications should not require the sand to be screened from the gravel and then mixed with the gravel again in fixed proportions which do not take cognisance of the proportion of voids in the gravel.

QUANTITY OF CEMENT.—One of the disadvantages of measuring cement and gravel by volume is the fact that the volume of a given weight of cement varies, according to the degree of consolidation. As the total cementing power depends upon the number of molecules, and as the number of molecules is proportional to weight and not to volume, greater uniformity would be obtained by determining the amount of cement by weight.

Recommendation.—That concrete specifications should be amended in such a manner that the amount of cement required is quoted as so many bags of cement per cubic yard of aggregate, and that a bag containing at least 200 lbs. of cement be taken as the unit.

QUANTITY OF WATER.—In some instances a difference of 1 per cent. of water may make a difference of 1,000 lbs. per sq. in. in the crushing strength of concrete at 4 weeks. In order to ensure the degree of flowability necessary for chuting the material it is necessary to have more water than is required to produce maximum strength. This practical difficulty has to be faced, and it is not therefore proposed to suggest any considerable change in the permissible stresses in concrete made of standard cement, but to approximate to the standard of stability adopted in the principal American codes.

BY-LAWS.—The by-laws made by the Council under Section 16 of the Metropolis Management and Building Acts Amendment Act, 1878, were confirmed in 1891, no less than 34 years ago, and there can be no doubt that they should be revised in several particulars. For example, the quality of Portland cement is referred to as though the British Engineering Standards Association did not exist (and it did not 34 years ago).

Recommendation.—That the necessary action be taken to obtain new regulations for concrete in substitution for the by-laws made by the Council under Section 16 of the Metropolis Management and Building Acts Amendment Act, 1878.

TENSILE STRESSES IN REINFORCEMENT.—In America, as here, a tensile stress of 16,000 lbs. per sq. in. is allowed in mild steel reinforcement. Where it is desired to include secondary stresses, however, it is recommended that the sum of the primary and secondary stresses should not exceed 20,000 lbs. per sq. in., with the object of allowing greater freedom in design whilst keeping the standard of stability at about the international standard which it is recommended should be aimed at. The regulations should be framed in such a manner that any designer wishing to adopt more scientific methods of design should not be penalised by having to include as primary stresses those secondary stresses which are usually supposed to be covered by the factor of safety.

RADIAL CONSTRUCTION.—In radial forms of construction the tensile reinforcement of floors is laid along lines radiating horizontally from the vertical axis of the pillars. There has been little or no demand for this type in England, and during the last twenty years no application has been received. It does not, therefore, appear necessary to make special regulations for a type of construc-

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tion which does not appear to be favoured by those who have had most experience of the requirements of London building owners. This form, however, is merely an instance of a departure from a standard type, and as it appears to be desirable that the Council should have power to deal with special cases on their merits it should obtain power of waiver in respect of special forms of construction in reinforced concrete.

Recommendation.—That the Council should include a paragraph in the next General Powers Bill to ensure that the power of waiver as given by Part VII of the London Building Act, 1894, in respect of brick buildings and temporary buildings, and as given by Section 5 of the London County Council (General Powers) Act, 1923, in respect of steel frame building, be extended to reinforced concrete building.

Hollow Reinforced Concrete Floors.—The London Building Acts require fire-resisting floors to have at least 5 in. of solid material. Hollow floors are, however, coming into general use, and are quite common in America. Section 336 of the New York Code is to the effect that when hollow burnt clay blocks are used in reinforced concrete floor construction the reinforced concrete between the hollow burnt clay blocks shall be designed to take the entire loads; provided, however, that when the overall volume of the hollow blocks does not exceed 60 per cent. of the overall volume of the floor construction not more than 2½ in. of concrete shall be required over the top of the hollow burnt-clay blocks. There is at present no such provision in the London Building Acts.

Recommendation.—That when hollow burnt-clay blocks are used in reinforced concrete floor construction which is required to be fire-resisting, the reinforced concrete between and over the hollow blocks shall be designed to take the entire loads, and the thickness of concrete over the crown of the hollow burnt clay block shall not be less than 2 in. A concession might, however, be made in cases in which the gaps or header joints between the ends of the hollow blocks can be filled up with a solid tongue or wedge of concrete, when perhaps some portion of the crown of the hollow burnt clay block might be regarded as assisting in the transmission of the compressive stress.

Coloured Concrete.—In the United States coloured concretes have been used for public buildings and houses with good effect. Portland cement is now made in a variety of colours, and the introduction of colour into concrete by using these cements promises to be a successful means of giving variety to concrete buildings. Experiments with white and coloured rough cast are to be made on the concrete houses at the Becontree housing estate.

Cast Stone.—The use of waste Portland stone crushed to sand sizes and mixed with Portland cement to form cast stone is a very useful and pleasing material, which is extensively used in the United States and should be given more attention in this country.

Considerable space is devoted to a discussion on town planning, and the conclusion is arrived at that any increase of the height of buildings in London should be only slightly in excess of the heights already permitted in suitable cases under the discretionary powers vested in the Council under the London Building Act, 1894; in the most favourable cases it might be possible to recommend a total height of 120 ft.

The report, which contains over 109 foolscap pages and 35 drawings, may be obtained from Messrs. P. S. King & Son, Ltd., price 5*s*.

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Fig. 1.—General View.

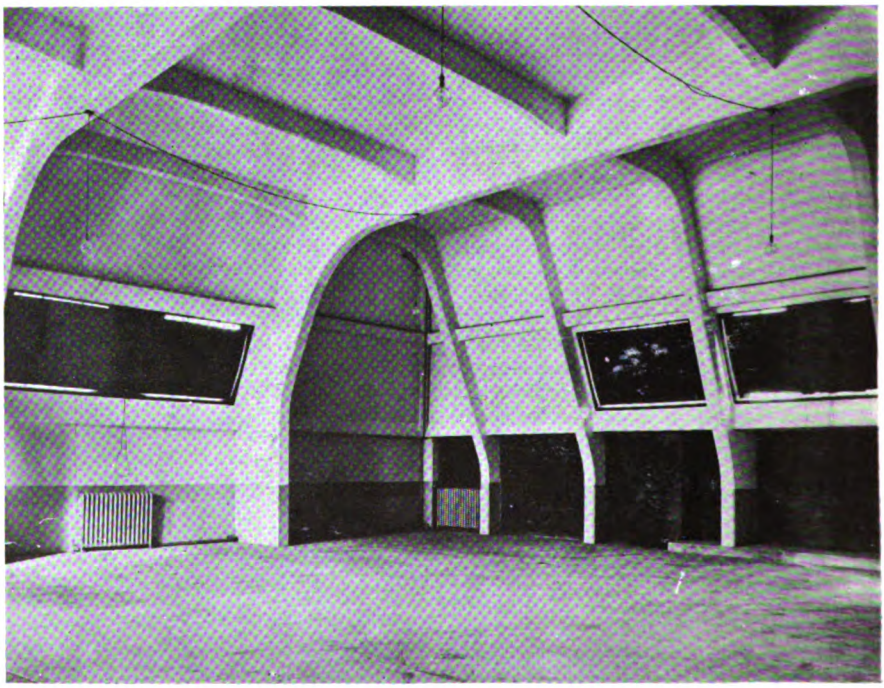

Fig. 2.—Gymnasium, showing Arched Roof Trusses.

The Frances Reckitt Institute, Hull. (*See p.* 337.)

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The Frances Reckitt Institute, Hull.

THIS Institute has been built as a social club for the employees of Messrs. Reckitt and Sons, of Hull, out of Trust Funds bequeathed by Mrs. Frances Reckitt. It comprises a large assembly room with concrete platform on the ground floor, basement with kitchen, etc., and three floors above ground floor used as work rooms, gymnasiums, etc.

The centre portion of the ground floor is covered by a glazed roof, carried by steel trusses, and extends the full height of the building. The side portions are covered by the first, second, and third floors, one side being for men and the other for women employees. The external walls of the building are of brickwork, the whole of the interior being of reinforced concrete; including columns, beams, floor slabs, roofs, etc.

The floors to gymnasiums at third-floor level are of hollow tile, so as to render them sound-proof. Both gymnasiums have a clear floor area of 56 ft. × 48 ft. and are 19 ft. high. There are no columns or other obstructions in the floor area, the roofs being carried across the 48-ft. span by specially designed trusses; these are in reinforced concrete, and details of the trusses are shown in *Fig.* 6. The trusses are fixed at floor level to the reinforced concrete floor girders at third-floor level. The large beams carrying these trusses are 46 ft. span in the clear, and also carry the steel trusses over the centre part of main assembly room. The combined dead and live loads, both distributed and concentrated, on these beams exceed 100 tons; details of these beams are shown in *Fig.* 7. A similar beam, but of 40 ft. span, carries the superstructure over the stage, at third-floor level. All beams are fixed at ends, bearing on the columns, and supported freely where resting on the brick walls. Specially designed reinforced concrete templates were used to distribute the loads on the walls. The columns and footings are in reinforced concrete, the reinforcement being round mild steel rods. The beams, floor slabs, and roof trusses, with the exception of the three long-span main beams, are reinforced with twisted steel bars. There are three staircases in the building, also constructed in reinforced concrete.

The concreting was done with a specially-designed concrete chute erected outside the building, and having a chute with a radius of 70 ft. All the concrete was mixed by machine and hoisted and tipped automatically into the chute.

The floors to the assembly hall, gymnasiums, etc., were designed to carry 2 cwts. per foot super.

The architect of the building is Mr. G. B. Carvill, of Trafalgar Buildings, Charing Cross. The general contractors were Messrs. G. E. Wallis and Sons, of Maidstone.

The reinforced concrete work was designed by Mr. T. A. Harvey, consulting engineer for Messrs. F. A. Norris & Co., who were the contractors for the reinforced concrete work.

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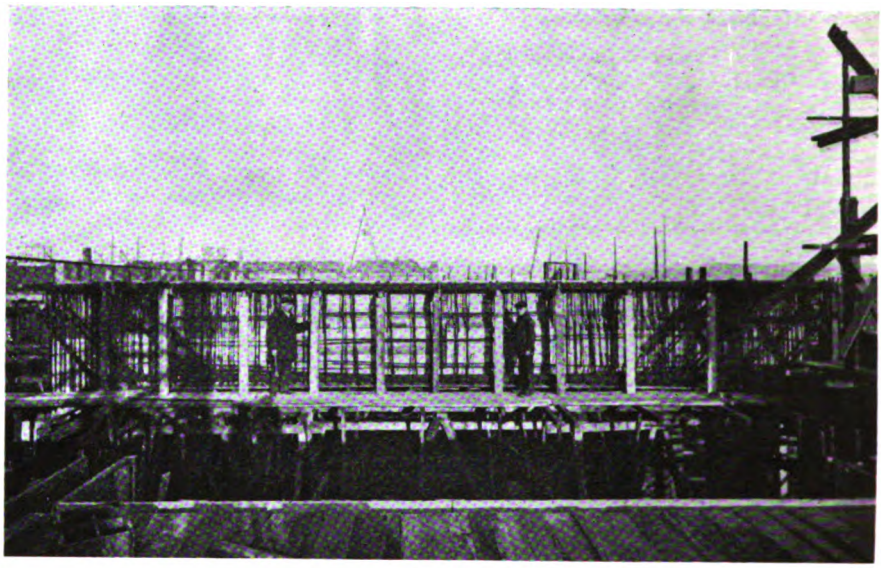

Fig. 3.—Shutting and Reinforcement of 46 ft. Span Beam in course of erection.

Fig. 4.—Floor Beams at First Floor Level.

The Frances Reckitt Institute, Hull. (*See p.* 337.)

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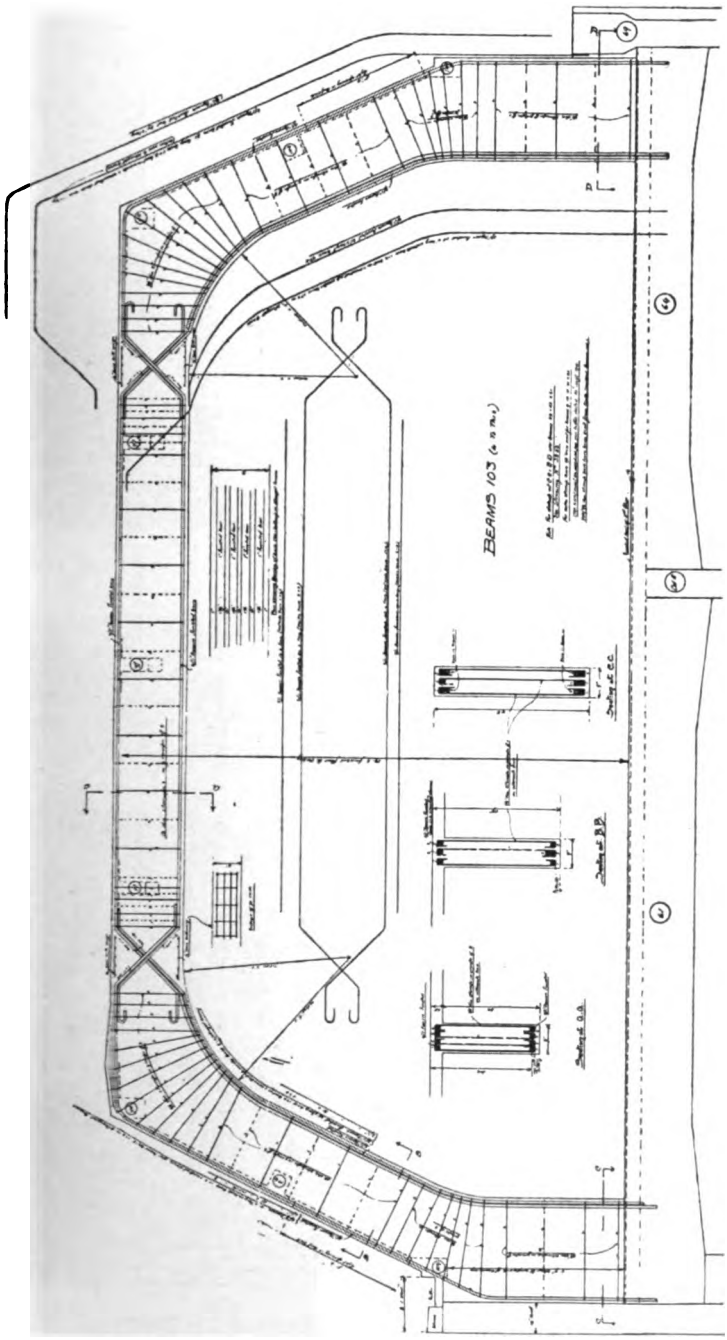

Fig. 6.—Detail of 48 ft. Span Trusses over Gymnasium. The Frances Reckitt Institute, Hull. (*See p.* 337.)

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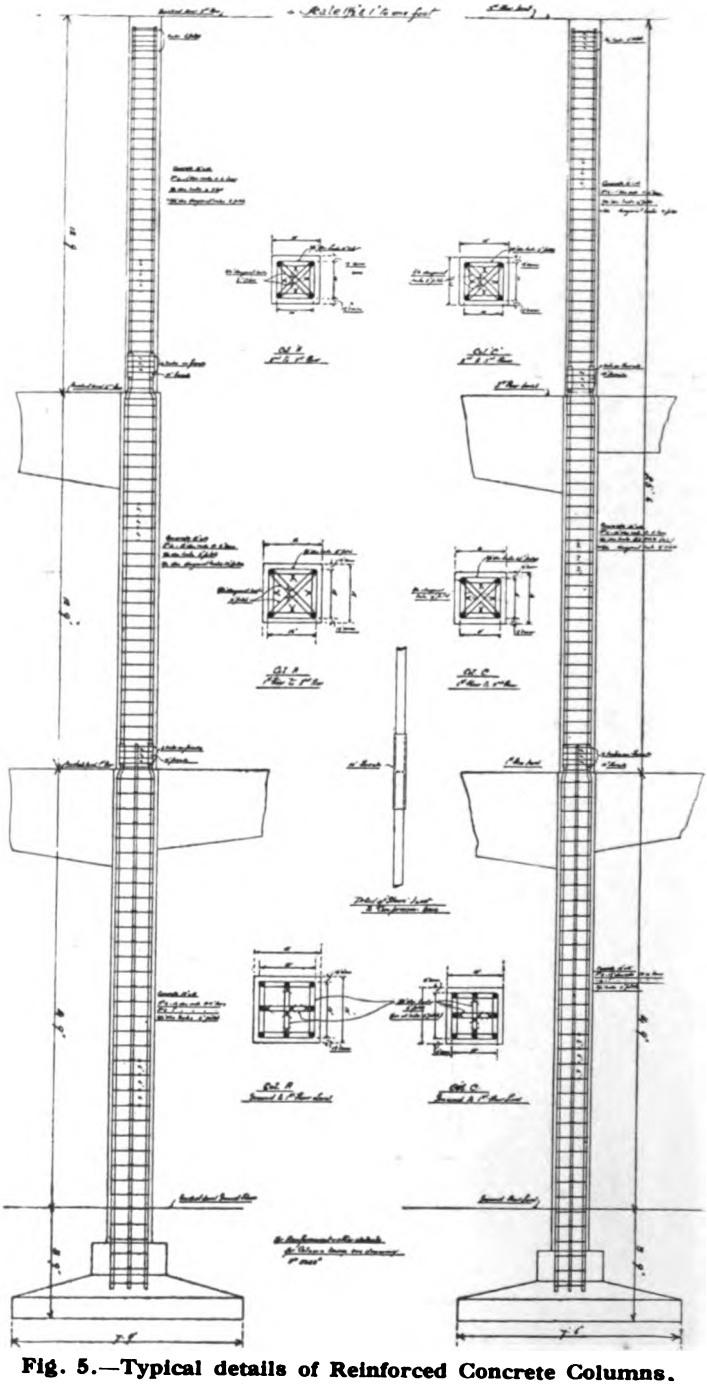

Fig. 5.—Typical details of Reinforced Concrete Columns. The Frances Reckitt Institute, Hull. (*See p.* 337.)

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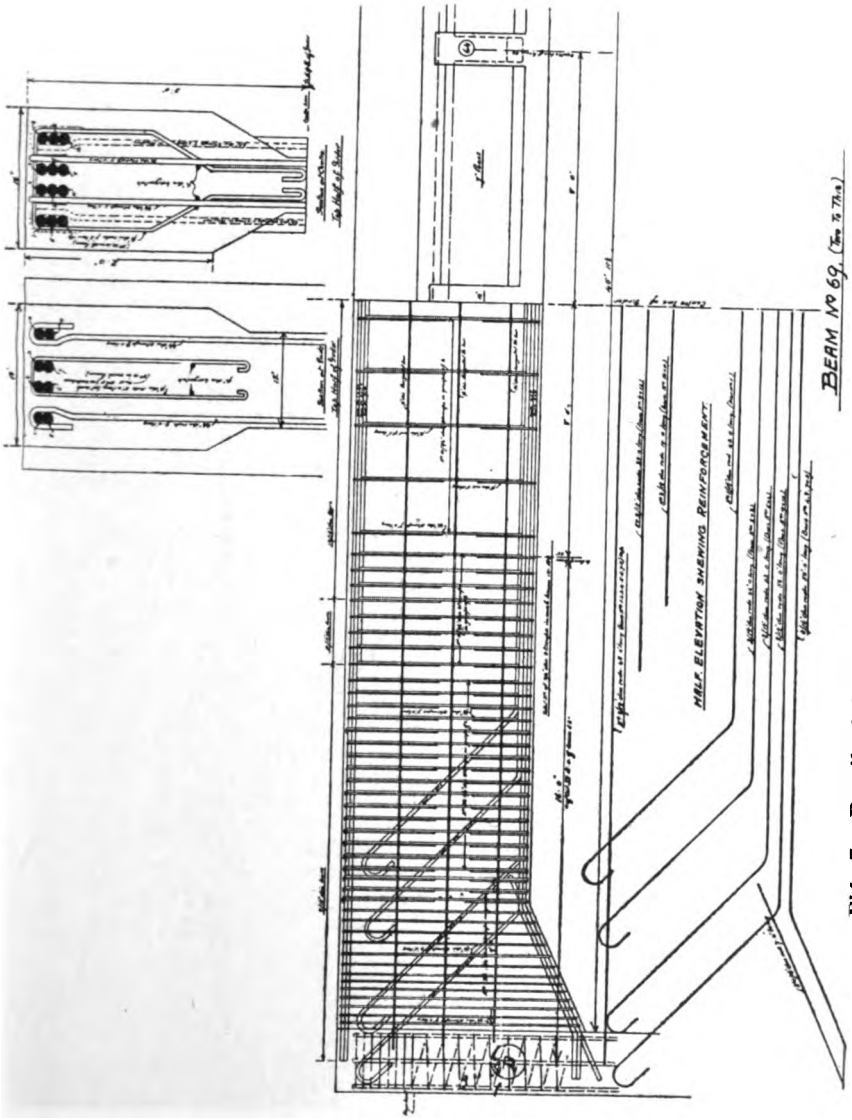

Fig. 7.—Detail of 46 ft. Span Beams carrying Roof Trusses.
The Frances Reckitt Institute, Hull. (*See p.* 337.)

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Specifications for Concrete Products.

We have received the following communication from a correspondent:—

Sir,—The makers of concrete building units sometimes receive, from persons who should know, specifications for concrete and reinforcement which are at variance with the accepted principles of practice and theory. It lies with the maker tendering for such work to cost and supply, if possible, exactly to the specification, or to take the risk, if receiving the contract, of doing what he may know to be better. He knows, or should know, that any failures in regard to stability or appearance will be blamed to the block-maker and not the specification. Unfortunately the block-maker dare not object to a specification if he wants the job, as any alterations he may suggest are often attributed to a desire to increase profits. Also, the powers that be do not care to have their dictum questioned. This being so, I shall be glad to have your opinion of the following clauses of a specification.

"The concrete for making blocks is to be composed of ballast crushed to a gauge not exceeding ½ in. sufficient clean sharp sand to fill the interstices of the aggregate and cement in the proportion of 6 vols. of ballast and sand to 1 of cement. The blocks, 18 in. by 9 in. by 6 in., (9 in. on bed), to have a 2 in. backing of clinker concrete composed of 6 vols. of clinker to 1 of cement. A richer mixture to be used for facing—to be composed of 1 vol. of cement to 4 vols. of sand. 9 in. by 9 in. lintels reinforced with two ⅞ in. dia. mild steel bars and approved stirrups or other bars placed 2 in. from underside and turned up at ends; include painting steel bars with one coat red lead paint. 10 in. by 9 in. lintels ditto ditto with three ⅞ in. bars and ditto."

Outside the question of the richer mix and the painting of bars is the percentage of steel, which is enormous unless the authority calculates transferring some of the compression to the solid and long blockwork above! No wonder concrete work is esteemed costly!

"Sarquem."

[We can appreciate the difficulty of concrete unit manufacturers when the specification for the work is not in conformity with the accepted principles of practice and theory. This difficulty, however, is not confined to concrete work but is met with in all classes of work in the building industry, and the particular—and sometimes peculiar—ideas of the client have to be handled in a diplomatic manner by the architect, and the particular fads of the latter have to be considered by the contractor, and so on. Individual designers have their own ideas on the subject of concrete work, and if these ideas are not based on sound experience or generally accepted principles the responsibility for failure must rest with them alone.

The only remedy in the hands of the manufacturer is to tender for the work as specified, and if necessary give an alternative price for the work in accordance with good practice with a covering note as to the reasons for the procedure. In the event of the order being received for the work on the lines of the original specification, and assuming this specification is not going to result in satisfactory work, the manufacturer should, before commencing any work whatever, write a letter to the "powers that be" stating that no responsibility can be accepted, and amplify this with straightforward reasons based on actual experience. The authority from whom the order emanated cannot reasonably object to this attitude on the part of an expert provided the letter is written in the right spirit and with all due respect to the feelings of the party to whom it is addressed. In any case the manufacturer must consider his reputation, and it is far better to lose the work than execute it on lines which will result in failure and bring discredit on all those connected with it.

With regard to the clauses of the specification quoted, it is always somewhat unwise to express a very definite opinion on a subject of this kind when the whole of the circumstances are not known. On the face of it, however, the clauses quoted are extremely unsatisfactory and as presented they indicate a poor knowledge of the subject, as the mixture specified for the facing would mean a poor job, the painting of the steel bars is contrary to the principles of good work, and the percentage of the reinforcement is from three to three and

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one-half times the economical amount. It is essential that the writer of a specification for concrete work should have a theoretical and practical knowledge of the subject, and we can only hope that the attempts which are being made to educate those in the building industry generally will, in time, result in a satisfactory and recognised standard specification for all classes of concrete construction.]

New Studies of Portland Cement.

In a recent paper, Mr. T. Merriman, Chief Engineer of the New York Water Supply, shows the result of an interesting series of investigations on the behaviour of cement with various solutions. He has devised a test which consists in agitating 10 g. of cement with 100 c.c. of a 15 per cent. solution of sugar for two hours, filtering and then titrating 25 c.c. of the filtrate with N/2 hydrochloric acid and phenolphthalein. He proposed to term the volume of acid so used the "solubility" of the cement, though this abnormal use of a common word is perhaps unfortunate. He clearly recognises that this "solubility" is due almost wholly to free line in the solution, and as calcium carbonate is insoluble in a solution of sugar, any calcium carbonate present in the cement will not be shown by this test.

So far the test is interesting, but no more. Everyone who knows much about Portland cement is aware that it is decomposed by water, and that one of the products is free lime, which is readily soluble in a solution of sugar, so that all this test shows is the amount of lime which can be dissolved in two hours by a 15 per cent. solution of sugar.

Mr. Merriman next proceeds to compare the tensile strength of the various cements after they have been mixed with 43 per cent. of water so as to form a slurry; this is poured into a mould and the resultant briquette is tested after storage under the usual conditions. The results seem to show that at 28 days the tensile strengths are highest with cements of high "solubility," but that the reverse is true at 70 days. When only 15 per cent. of water was used in making the briquettes all the cements with high "solubility" were weak from 7 to 28 days. The suggestion is that by using 43 per cent. of water the skill ordinarily needed for making cement tests is avoided—a very serious mistake, as the very harmful effect of even a small excess of water on the strength of cement, and still more on that of concrete, has been proved. The sole reason for using the small amount of water ordinarily employed in making cement tests is that it produces the best results that can be obtained; to use more water is to spoil the cement and reduce its tensile strength. Hence the use of a large excess of water—no matter how convenient—is very harmful and should be avoided.

Mr. Merriman's "solubility" test has no direct connection with the strength of the cement; what it does show is the rate at which the cement is decomposed by the sugar solution. If the cement has been aged (either naturally or artificially) it will decompose more slowly than a fresh quick-setting cement. It is, however, quite wrong to suppose that a cement with a low "solubility" in sugar solution is necessarily either inferior or superior to one with a high "solubility," as so much must necessarily depend on the purpose for which the cement is to be used. For this reason Mr. Merriman's suggestion (*Engineering News-Record*, 1924, p. 672) that cement manufacturers should grind and store their product in an atmosphere of carbon dioxide in order to reduce its "solubility" is particularly harmful, as it is merely slowing down the decomposition of the cement whilst in use, and is therefore retarding the development of the strength of the concrete. Moreover, the portion of the cement which is carbonated is of very small value as a cement and must be regarded at the best as so much inert material.

The proposed recommendations, based on a misunderstanding of the solubility of cement in a saccharine solution, are singularly unfortunate, and whilst it is unlikely that British cement manufacturers will adopt them, it is important that waterworks engineers and others should not be misled by such a test in comparing different cements.

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Questions and Answers relating to Reinforced Concrete.

Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.—ED.

Bow-String Bridges.

QUESTION.—I would be obliged if you would indicate the method of stress analysis with particular reference to proportioning of pure tension members in reinforced concrete bow-string bridges. I have been unable to obtain this information in any of the books with which I am acquainted, beyond the fact that in one case it has been mentioned that the main members have been treated as an eccentrically loaded column.—A.W.F.

ANSWER.—A parabolic bow-string bridge with vertical tension members and no bracing can be designed as regards the main stresses in both the bow string and the tension member as a pure arch from which the deck is slung. The tension members at the bottom are, of course, designed to resist a tension equal to the thrust of the arch. The tension members at the bottom also act as beams spanning from one panel point to the next, and therefore have to be designed for the combined stresses. Where the bridge is liable to unequal loading there will in addition to the main stresses be considerable bending stresses which will be resisted by the top and bottom members as well as the vertical members. The calculation of these racking stresses is rather outside the scope of what can be dealt with as a reply to a question.—O.F.

Mortar.

QUESTION.—I have built a concrete block warehouse, and find the mortar at the seams cracks and strikes damp in parts. Could you tell me what mixture of pointing material would be best to use to remedy this fault on this class of structure ?—J. E. I.

ANSWER.—The trouble would appear to be due to foreign substances in the pointing material, and if the mortar has been mixed with unwashed sand the defect can probably be traced to this. The remedy suggested is to rake out all defective material and repoint with cement mortar made with three parts of good sharp well-washed sand and one part Portland cement obtained from a reliable manufacturer.—A. L.

Concrete Floors.

QUESTION.—What is the most used finish for reinforced concrete floors on sitting, dining, and bedrooms ? What finish is preferable for the same purpose : (*a*) the cement coat covered with linoleum or (*b*) jointless compositions ? Are there any good formulæ for jointless composition floors that could be applied in places where there are not special firms ?—M. T. V.

ANSWER.—The most economical finish for reinforced concrete floors will be that composed of granite chippings and Portland cement about one inch thick, and known as granolithic. It is, of course, possible to lay tiles, wood blocks, or almost any kind of flooring according to the requirements and the money that can be expended. Generally speaking a granolithic floor covered with linoleum is more reliable than a jointless composition flooring owing to the uncertain nature of the magnesite which is one of the principal ingredients of the latter. A formula for a jointless flooring that can be used in districts where no specialist firms are available is as follows :—Two parts of sawdust to one part of ground magnesite by volume are mixed thoroughly together and magnesium chloride (1·18 to 1·19 specific gravity) is added in sufficient quantity to moisten the mixture. The materials are then laid and pressed down and ironed over with a steel float. If any particular colour is required a pigment in powder form is added to the dry materials and thoroughly incorporated before the magnesium chloride is applied. The ground magnesite, if not perfectly burnt, is liable to cause defects to occur, and for this reason there is always a risk in this type of flooring, as unfortunately it is impossible to detect any fault in the magnesite by test or examination before use.—A. L.

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The Foreman's Guide to Concrete.—VIII.

A MANUAL FOR ALL ENGAGED IN CARRYING OUT CONCRETE AND REINFORCED CONCRETE STRUCTURES.

By Albert Lakeman, M.S.A., M.I.Struct.E.

BENDING MOMENTS (*Continued*).

BEFORE completing the notes on the variation of bending moments, it is advisable to indicate the effect of restraining or fixing the ends of the beam, because this is a condition which so often occurs in practice. It was mentioned, when dealing with continuous beams, that the portion on each side of the support acted as a cantilever while the central portion acted as a beam that was carried by the ends of the cantilevers, and this caused tension stress on the upper and lower surfaces alternately, as illustrated in *Fig.* 40.

The point at which the stress changes is known as the point of contraflexure, and where this occurs there is no tension or compression stress; consequently the ordinary simple rules given for the leverage of reactions and loads in the case of supported beams cannot be applied to continuous beams. If there is no stress it indicates that there can be no bending moment, and yet there must be a bending moment at the support because the cantilever portion is called upon to carry the ends of the central beam portion.

These conditions mean that there is a bending moment at the centre of the span, a bending moment at the support, and no bending moment at the point of contraflexure. The reasoning and the calculations necessary to show why and how the value of the bending moment is ascertained are too advanced for these notes, and it will be sufficient if some idea is given of the variation at the different points. A continuous beam carrying a uniformly distributed load is shown in *Fig.* 57, in which the shaded diagram below the beam indicates the variation in the bending moment. The line *A–B* is a horizontal line which separates the bending moment curve into two portions, that above the line indicating the moment in the beam portion, and that below the line showing the moment in the cantilever portion where the tension stress occurs on the upper surface. It will be noticed that the bending moment curve crosses the horizontal line *A–B* at the point of contraflexure, and as the value of the bending moment at any point equals the length of the vertical line between the curve of the line *A–B*, it will be clear that where these two meet no moment can exist. The curve is a parabolic curve similar to that given in *Fig.* 56, but it is cut into the two portions by the line *A–B*. It will be seen that the maximum bending moment at the support gradually diminishes towards the point of contraflexure, where it is nil; and from this point it gradually increases towards the centre of the span, where it again reaches the maximum. The maximum reinforcement is therefore required at the support and at the centre of the beam where the load is a uniformly distributed one and the beam is continuous over several spans or is fixed at the ends, and as the tension stress changes from the bottom of the

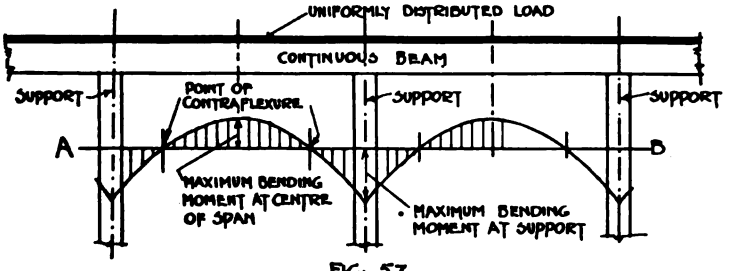

FIG. 57

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beam to the top, so the rods are cranked up from the lower surface to the upper.

Many complications may arise when calculating fixed and continuous members in a reinforced concrete structure, but the consideration of difficult cases would only tend to confuse the reader and involve the use of formulæ that can be found in text-books devoted to the design of reinforced concrete. The remarks given when dealing with the description of continuous beams should be referred to after the diagram in *Fig.* 57 has been carefully studied, and the necessity for executing this type of member in accordance with the notes previously given will be appreciated.

Shearing Stress.

An explanation of shearing stress was given when dealing with the different types of stress that occur in building work,

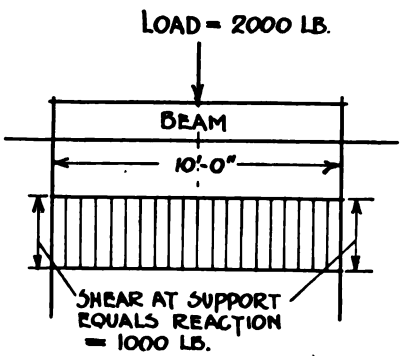

FIG. 58.

but some further notes showing the variation throughout the length of a beam are necessary in order to make the explanation complete.

The shearing stress is caused by the load and reaction which oppose each other through the agency of the beam, as the load tries to push downward and the reaction is an upward resistance. The tendency in the beam is for portions to slide past one another both vertically and horizontally; the question of leverage does not arise as in the case of bending moments because the stress is caused by direct opposition of two forces acting in different directions at all points in the beam. The value of the horizontal shear and the vertical shear at any point will be equal, or in other words the tendency for the beam to slide in horizontal layers is the same as the tendency to slide into vertical layers, and therefore the use of

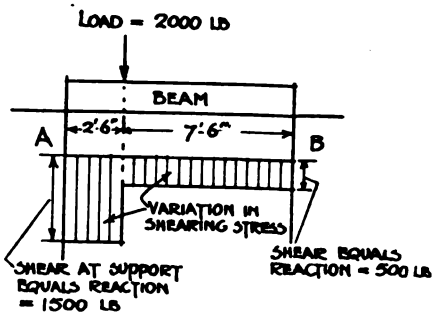

FIG. 59

the term "shearing stress" and the value given in connection with any particular example means either or both types of shear.

As the shearing stress is caused by the opposition of the reaction to the load, the maximum shear will always occur at the supports when the shearing stress equals the value of the reaction. A simple example is given in *Fig.* 58, where a beam carries a central concentrated load of 2,000 lbs. over a span of 10 ft. As the load is central the reaction at each support will equal 1,000 lbs. and the shear at this point will also be 1,000 lbs. The amount of the load travelling from the centre point to the abutment in each direction is 1,000 lbs., and the shearing stress at all points will therefore be the same; the shaded portion of *Fig.* 58 therefore indicates the stress at the various points throughout the length.

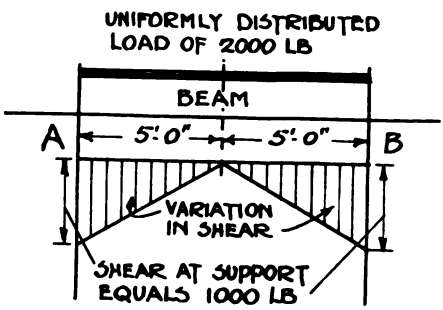

FIG. 60.

If the load is concentrated at a point which is not central, as in *Fig.* 59, the shear will be greater at the support which

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is near to the load, as the reaction will be equal to more than one-half the total load. In the example the load of 2,000 lbs. is situated at a distance of 2 ft. 6 in. from one support and 7 ft. 6 in. from the other, and the reaction at the support *A* will therefore be 1,500 lbs., while at *B* it will only be 500 lbs. The maximum shearing stress will therefore be equal to 1,500 lbs. at any point between the load and the support marked *A*, and the stress between the load and the support *B* will equal 500 lbs. at any point. This variation is again indicated by the shaded portion of *Fig.* 59.

In the case of uniformly distributed loads the variation in the shearing stress is somewhat different, and this is shown by the shaded portion of the diagram in *Fig.* 60. The maximum shear will be equal to the reaction at the support, which in this example is 1,000 lbs., but the shear will gradually diminish to the centre of the span, where it is nil. There should be no difficulty in understanding this, because as the load travels along the beam towards the support it gradually increases in value, and the shear at any point equals the amount of the load passing along the beam at that point on its way to the support. As the beam has a span of 10 ft. and the total load is 2,000 lbs., the load per foot run is 200 lbs. At a point 1 ft. from the centre, therefore, the load travelling to the support is 200 lbs., whereas at 2 ft. from the centre the load has increased to 400 lbs., and so on until the support is reached where the amount has become 1,000 lbs., which is the value of the reaction.

When the beam is carrying concentrated and distributed loads the shear will be equal to the combined shears due to each load, and as an example the reader is referred to *Fig.* 61, which is a combination of the central concentrated load shown in *Fig.* 58 and the uniformly distributed load given in connection with *Fig.* 60. The shaded portion indicates the shear at the different points in the length, the distance between the lines *AA* and *BB* showing shear due to the concentrated load, and between *BB* and *CC* the amount due to the distributed load. It will be seen that the maximum shear at the support is 2,000 lbs. and the shear at the centre is 1,000 lbs.

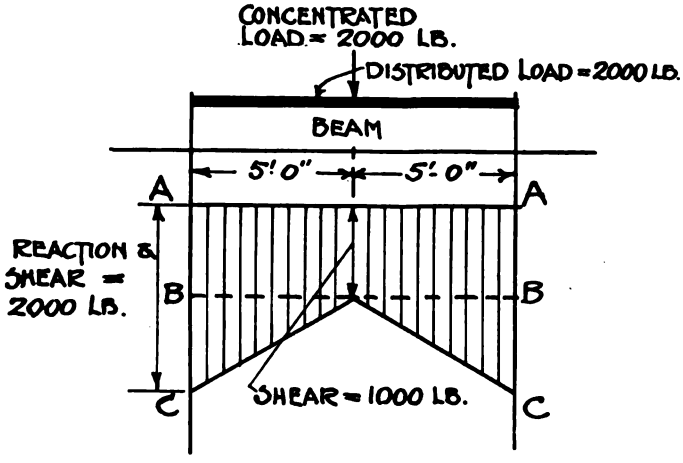

FIG. 61.

As a provision against the failure of a reinforced concrete beam by shearing, stirrups are introduced, and these are spaced according to the total stress that has to be dealt with. It is not proposed to give the methods of calculating the number and positions of the stirrups, as this would involve a detailed explanation of the distribution of the stress across the sectional area of the beam, and the application of theory which is outside these general notes. It can be stated, however, that a single reinforced concrete beam which is economically designed theoretically without any restrictions on the

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depth of the concrete will contain sufficient resistance to shear without the necessity of providing stirrups, but to comply with the regulations and in order to anchor the tensional steel to the compressional concrete some stirrups must be provided.

Where the shearing stress is in excess of that which can be taken up by the concrete, and stirrups or shear members are theoretically required, these are carefully calculated and the spacing is determined in accordance with the intensity of the stress. It has been shown that the greatest shear is at the supports, and this means that the stirrups will be closer together at the ends of the beam than in the central portion, and the reason for this will be realised when the reader has thoroughly grasped the explanations given.

of the stirrups and the care taken by the designer in calculating the sectional area of steel required, the number of stirrups necessary, and the correct spacing.

Stirrups are of different types, and flat or round steel may be specified, while some are vertical and others diagonal, and variations are met with as regards attachment to the main steel rods, and the method of placing either around a single rod or around a set of rods, but in every case the drawings should be strictly followed and no attempt made to make variations in the mistaken idea that some practical experience can be applied which will simplify the execution.

Slabs.

Sufficient general notes have been given to cover the principles of reinforced concrete beam design, and the particulars

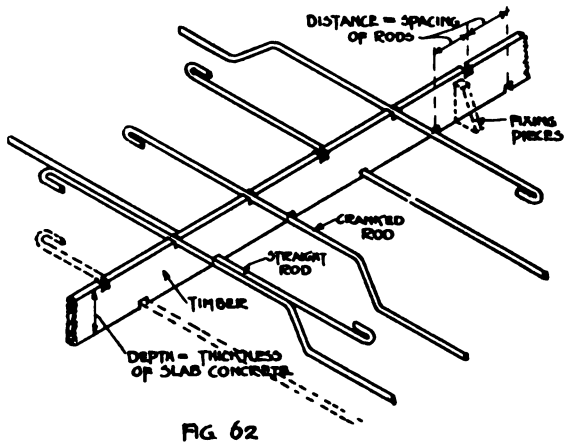

FIG. 62

The necessity for accurate spacing in the work and the exercise of care in placing the concrete to avoid the displacement of the stirrups will also be apparent, as each stirrup has been spaced on the drawings by the designer in a manner which will ensure a resistance that is equal to the shear at all points, and if the position of the stirrup is varied then the resistance will also be varied and the designer's skill be wasted. Instances have arisen where the stirrups have been found pushed together in a mass at two or three points in a beam due to carelessness in execution, and although such cases are happily very infrequent it is clear that they would not occur at all if the foreman in charge were conversant with the object

given will apply also to reinforced concrete slabs as regards the main points. But no complications will arise as regards shearing stress, bending moments due to combined loading, restrictions in depth as with tee beams, and other factors which have been dealt with, because the beams supporting the slabs will be arranged to allow the latter to be designed in an economical manner on the same lines as a single reinforced beam. The slabs may, of course, be supported only or continuous, and care must be taken to space the rods accurately and provide cranked rods where these are shown.

On work in progress where slabs are being concreted the author has often

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observed the difficulty experienced by workmen in keeping the steel accurately in position, particularly with regard to the rods which are cranked up, as there is a tendency for these to fall over sideways. The use of a piece of timber that is notched as shown in *Fig.* 62 will overcome this difficulty, and it has been successfully adopted on many schemes although the system does not appear to be generally followed. The notches are cut on both edges of the board according to the spacing of the rods, and it acts as a template which obviates the necessity for measuring the distance between the rods. It also makes inspection and control a simple matter, as the templates can be checked by the foreman or clerk-of-works in the first instance and incorrect spacing is then impossible. It does not matter from which side the rods cross the template, as shown in the diagram; the board is easily kept in position on the formwork, and acts as a stop for the end of the bay or panel which is being concreted. When the concrete is placed and it has hardened sufficiently the fixing blocks are removed and the board is pressed over into a flat position, when it can be withdrawn from between the rods without any difficulty and transferred to the end of the next bay for use again.

The small rods which are provided in slabs at right angles to the main reinforcement are distribution rods which act in distributing the load, but their principal function is to assist in preventing cracks due to the expansion and contraction of the concrete under changes of temperature. These small rods are not part of the calculated steel, but are rather a practical factor, and the size and spacing are dependent on the judgment of the designer. These distribution rods should be wired to the main rods at the intersections, and they will then assist in keeping the steel properly spaced during the placing of the concrete.

The importance of keeping the reinforcement in its proper position cannot be stated too strongly, especially as the point does not always seem to be realised by the workmen on the job. The author has noticed in many instances that the slab reinforcement is carelessly placed, and even when correctly put into position at the outset it is displaced by traffic over the steel; unless great care is exercised it is concreted in without proper adjustment. Slab rods are also frequently bent and distorted during the operations, and they are then sometimes roughly bent back to their position but left with kinks and distortions which prevent effective direct resistance to tension. One square inch of steel does not seem perhaps a very important factor in a building, but it should always be remembered that this represents 16,000 lbs., and a pull of this value is not an item to be lightly dismissed.

Again, one square inch of concrete hardly seems worthy of any serious comment, and yet this represents 600 lbs. and if it were an actual weight which was visible and had to be lifted it would command some respect. Holes are sometimes cut through reinforced concrete slabs for the passage of pipes, conveyors, and other similar purposes, and the workman is allowed to cut away the concrete and the steel rods without any regard to the effect on the strength of the floor. This is invariably due to a lack of knowledge on the part of the foreman who has insufficient technical training.

A full appreciation of the stresses that have to be dealt with both as regards nature and intensity will cause a supervisor to consider carefully the consequences before any cutting away is done, more especially if he is likely to be held responsible for any defect that may occur.

(To be continued.)

Advice on Concrete.

In pursuance of its policy of centralisation, the British Portland Cement Association has transferred several cement and concrete testing laboratories to their premises at Westminster. The Association is a non-trading body, and the new central organisation has been formed for advising on the applications of cement and concrete, and it is stated that the new laboratory is the finest of its kind in Europe.

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New Method of Cottage Construction.

A METHOD of building composite walls with the use of the well-known "Metaform" shuttering has been patented by Messrs. A. A. Byrd & Co., of 11, Queen Victoria Street, E.C.4. The system has been produced particularly with a view to completing the shells of dwelling-houses ready for paper or distemper without the services of either the skilled bricklayer or plasterer. A composite wall, composed of an impervious concrete for the outer part and a porous concrete for the inner face, is poured *in situ*, the two kinds of concrete being kept separate from each other during pouring by ordinary wire netting, the interstices of which permit the concrete to become firmly bonded together.

The material used for the inside of the wall is any suitable mixture which has a similar nature to plaster and is equally porous, whilst the material used for the outside of the wall is ballast concrete.

By moulding the wall with "Metaform" interlocking steel forms a smooth sealed face is produced so that when the forms are removed no plaster or making-up is necessary and the inside faces are ready to receive paper or distemper.

When a house is to be constructed on the new system the first operation after the footings and damp-course are completed is to lay out sheets of 2 ft. by ½ in. wire netting so that one selvedge is exactly over a line previously marked on the footing. Whilst laying flat this selvedge is fastened to the line with staples or spikes driven into the concrete. The next stage is to set-up and fix the first course of forms to the footing. As soon as the forms are in position special staybars, shown in *Fig.* 1, are placed across the edges of the forms and held in position by pins which fit into holes in the top angles of the forms. These staybars have a special clip carried in a bridge which grips the top running edge of the wire netting at three points, and the proper amount of tension on the netting is obtained by adjusting the thumb-nuts which raise or lower the clip as required. This ensures the wire netting being held flat and taut, and in its proper position vertically between the two rows of forms. The staybars may be spaced to give any desired total wall thickness, and the netting can be held to give any combined proportions of impervious or porous walls. When the netting is taut the wiring of the forms is done in the usual manner. The two different materials are poured at approximately the same level throughout the course, and tamped with a ⅜ in. rod.

Whilst the pouring of one course is proceeding the next course of netting is

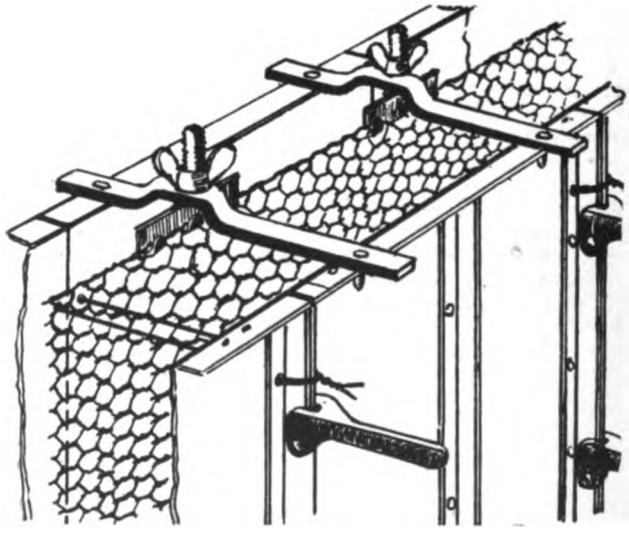

Fig. 1.

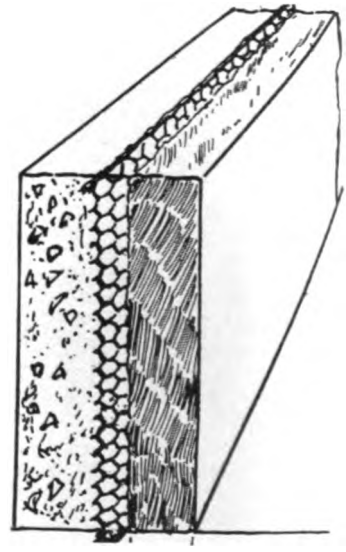

Fig. 2.

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set-up and laced to the top edge of the last course of netting with thin binding wire.

The forms are stripped by knocking down the clamps with a hammer and cutting the wire loops, when the forms are free to come away from the face of the wall. One man on each side of the wall removes and cleans each plate, brushes thin oil over the face, replaces it in its same relative position on the new course above, and inserts nails to keep it in position.

Window and door openings are stopped off by temporary wood frames the full wall width, which are inserted inside the shuttering so that the concrete is stopped off at the required places.

It is claimed that with this system the whole of the walls of a three-bedroom non-parlour type house, having a frontage of 19 ft. by 26 ft. depth by 16 ft. height of walls to eaves, based on 8 in. thick walls composed of 5 in. 4 : 2 : 1 ballast concrete and 3 in. porous material, can be completed at an inclusive cost of £90.

Rapid Hardening Portland Cement and Pre-Cast Beams.

An interesting series of tests has been carried out by the Siegwart Fireproof Floor Co., Ltd., on "Siegwart" beams made with "Ferrocrete" rapid-hardening Portland cement. The beams were made of a mixture of 1 part "Ferrocrete" to 8, 6, and 4 parts of $\frac{3}{8}$ in. shingle and sand respectively, the proportion of shingle to sand in each case being about 50 per cent. The design of the hollow beam is shown in *Fig.* 2. The beams were 9 ft. $4\frac{1}{2}$ in. long, 10 in. wide, and 5 in. deep, with a hollow core, and they were tested spanning across supports 8 ft. 8 in. apart. The full result of the tests is given in the following table :—

Gauged	Weight cwts.	Deflection. inches.
8 & 1	5	$\frac{1}{16}$
	10	$\frac{1}{2}$
	12	Broken at point of contraflecture.
6 & 1	5	
	10	$\frac{3}{16}$
	15	$\frac{1}{4}$
	20	$\frac{1}{2}$
	25	$\frac{7}{8}$

Hollow Reinforced Concrete Beam loaded with Two Tons four days after Casting.

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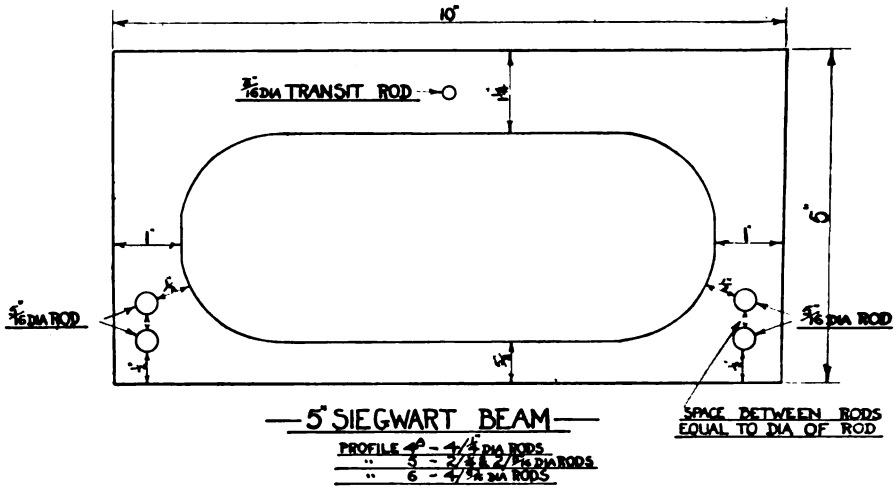

Gauged	Weight cwts.	Deflection. inhces.
6 & 1	29	Broken 2 ft. 6 in. from inner edge of support. Failed on shear.
4 & 1	10	1/16
	15	1/4
	20	5/16
	25	3/8
	30	3/8
	35	7/8
	40	1 1/4
		Cracked in centre. Failed on tension.

The age at testing was 96 hours, and the Siegwart Company inform us that these results are about 12½ per cent. better than with similar beams made with ordinary Portland cement and matured for three weeks. The "Ferrocrete" beams were designed for a superposed load of 1¼ cwts. per sq. ft, with a factor of safety of 4, and the actual breaking load obtained was nearly 6 cwts. per sq. ft.

A further test on one of those beams was undertaken to ascertain the strength when the concrete was mixed with less water. A 4 : 1 concrete was used, mixed with 20 per cent. less water than the specimens of which the test results are given above, and it was found that at all stages of the test a greater strength was shown. There was no deflection at all with a load of 10 cwt., and a total load of 45 cwt. was applied before the beam broke in the centre.

Concrete Hoardings.

THE collapse of 120 ft. of hoarding near Waterloo Bridge has again drawn attention to the danger of these structures unless properly erected. Their weight is frequently considerable, and their collapse upon a crowded pavement might cause serious loss of life. In order to avoid such mishaps, and also for reasons of economy, the London Midland and Scottish Railway recently introduced a standard type of construction for their hoardings. The structure supporting the actual woodwork, to which the posters are affixed, consists of a concrete base from which rise posts of reinforced concrete. "The number of the Company's hoardings is immense, with a display area totalling many square miles." an official at Euston has stated, "and not one has fallen down since we began to use concrete posts and bases. Besides the advantage of added safety, the cost of maintenance will be a mere fraction of that incurred under the old system."

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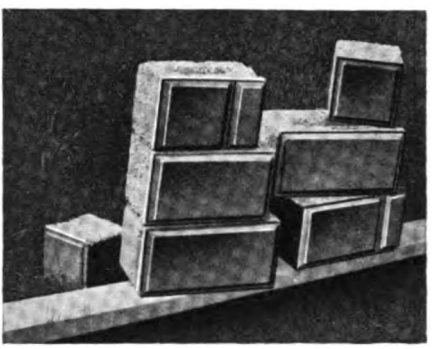
New Type of Concrete Block.

THE problem of obtaining a concrete wall with a weatherproof outer face and an inner face which will absorb the moisture from the atmosphere and thus prevent condensation has resulted in the advocacy and use of many different systems. The latest attempt of solving the difficulty to be brought to our notice is the "Trinity" hollow block, patented by Capt. J. Brunton, of "Leeside," The Avenue, Welwyn, Herts, the novelty of which lies in the use of a course of bitumen about 2 inches from the face. The outer face of the block is made of dense gravel concrete, and behind this is a sheet of bitumen which absolutely prevents any possibility of damp reaching the breeze concrete which forms the backing and inner face. For reasons of economy in materials and to keep down the weight the block has a hollow vertical core. The size of the standard block is 18 in. by 9 in. by 9 in., and three-quarter and half size blocks and quoins are used to facilitate following any design and to break joint between the courses. Thus while the breeze backing prevents condensation and is a poor conductor of heat and cold, keeping the house warm in winter and cool in summer, the bitumen makes the block completely waterproof from the outside. The cost of the block is also reduced, as the cheaper material, breeze, preponderates, and the layer of bitumen is incorporated during the process of manufacture without appreciably adding to the time taken in making the blocks. The face of the blocks can, of course, be of any desired pattern or colour.

A further feature of these blocks is the use of "Ferrocrete" rapid-hardening Portland cement, which allows of the blocks being used within two or three days of manufacture, thus avoiding possible delay in delivery and also eliminating the necessity for a large area for storage while they are curing. The house in course of construction which we illustrate is being built at Welwyn, and architects and others interested are invited to inspect it by appointment.



The "Trinity" Concrete Block.

House in course of construction at Welwyn built with "Trinity" Concrete Blocks.

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Thames ballast	,,	10	6
Broken brick (¾ in.)	,,	11	6
Best British Portland Cement	per ton 58s. to	63	0
"Ferrocrete" Rapid-Hardening Portland Cement	delivered London 10s. per ton extra		
"Super-Cement"	per ton 88s. to	93	0
"Lightning" Brand Aluminous Cement	,, 100/- f.o.r. works.		
Ciment Fondu	130/- ex stores		

BOARDING FOR SHUTTERING—		Sawn. s.	Sawn. d.	Wrot. s.	Wrot. d.
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in. . . . . from £23 per standard
3 in. by 6 in. and 3 in. by 7 in. . . . . ,, £23 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	0
9/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds	,,	14	6
¼ in. Rounds	,,	15	6

Breeze Slabs per yd. super: 2 in., 1/11; 2½ in., 2/4; 3 in., 2/9; 4 in., 3/6.

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 : 2 : 4—		s.	d.
Do. do. in foundation	per yard cube	48	6
Do. do. in columns	,, ,,	53	6
Do. do. in beams	,, ,,	53	6
Do. do. in floor slabs 4 in. thick	per yard super	5	8
Do do. in floor slabs 6 in. thick	,, ,,	8	5
Do. do. in floor slabs 9 in. thick	,, ,,	12	3
Do. do. in walls 6 in. thick	,, ,,	8	8

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—

		s.	d.
From ¼ in. to ⅝ in.	per cwt.	25	0
,, 9/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2⅞ in.	,,	23	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1½*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2½*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.* (per bend per cwt.).

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.* (per bend per cwt.).

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	,, ,,	1	1½
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1½
Do. do. as last, in narrow widths	,, ,,	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Carpenters working on old shuttering, 1/10½; Labourers on building works, 1/4½; Men on mixers and hoists, 1/5½; Bar-benders, 1/5½.

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Prospective New Concrete Work.

ADWICK-LE-STREET. — *Houses.* — The U.D.C. proposes to erect 300 houses.

ALDERSHOT.—*Houses.*—The T.C. proposes to erect 100 houses.

ARDROSSAN.—*Houses.*—The T.C. proposes to erect 48 houses.

ASHINGTON. — *Road.* — The Morpeth R.D.C. and the Ashington U.D.C. are considering a scheme for the construction of a road from Ashington to Ellington. The estimated cost is £44,000.

BALLYSHANNON. — *Breakwater.* — The Harbour Commissioners are seeking a loan towards the cost of erecting a breakwater, estimated to cost £500,000.

BATHGATE.—*Reservoir.*—The D.C. proposes to construct a reservoir, estimated to cost £200,000.

BETTWS.—*Houses.*—The St. Meelon's R.D.C. has appointed a sub-committee to inquire into a proposed scheme for the erection of 1,000 houses.

BOLSOVER.—*Houses.*—The U.D.C. has applied for sanction to borrow £21,000 for the erection of 62 houses in Moorfield Lane.

BRENTFORD. — *Houses.* — The U.D.C. has purchased land for the erection of 170 houses.

BRENTWOOD — *Houses.* — The U.D.C. proposes to erect 100 houses.

BRIGG.—*Houses.*—The U.D.C. is negotiating for a loan of £20,000 for the erection of 46 houses.

BUCKHAVEN.—*Houses.*—The T.C. proposes to erect 80 houses.

BURTON-ON-TRENT. — *Houses.* — The Corporation has notified the M.H. of its scheme for the erection of 298 houses.

CAMBUSKENNETH.—*Bridge.*—The Stirling C.C. proposes to erect a two-span bridge across the Forth.

CARMARTHEN.—*Bridge.*—The Carmarthenshire C.C. is to erect a new bridge in place of the existing structure.

CHELMSFORD.—*Houses.*—The T.C. has applied to the M.H. for sanction to erect 250 houses.

CHESTERFIELD.—*Houses.*—The T.C.'s housing scheme provides for the erection of 300 houses during the next two years.

CLYDEBANK.—*Houses.*—The T.C. proposes to build 146 houses, estimated to cost £55,000.

COLNE.—*Houses.*—The T.C. has applied to the M.H. for sanction to erect 60 houses on the Haverholt estate.

CONISBROUGH.—*Houses.*—The U.D.C. has applied for sanction to borrow £164,598 for housing purposes.

CONSETT.—*Houses.*—The U.D.C. has applied for permission to erect 300 houses.

CWMCARN. — *Houses.* — The Abercarn U.D.C. is considering a scheme for the erection of 150 houses.

DERBY.—*Concrete Houses.*—The Borough Surveyor is inquiring into the cost of erecting 100 concrete houses.

DUNSCROFT. — *Houses.* — The Thorne R.D.C. has under consideration a scheme for the erection of 250 houses at Dunscroft.

EBENEZER.—*Reservoir.*—The Gwyrfai R.D.C. has applied to the M.H. for sanction to borrow £1,600 for the erection of a service reservoir at Ebenezer.

EDINBURGH.—*Houses.*—The Corporation Housing and Planning Committee recommends the erection of 500 houses.

EPSOM. — *Grand Stand.* — The Epsom Grand Stand Association proposes to raise new capital to the extent of £100,000 for the construction of the stands, which are to be erected mainly in concrete.

EPSOM.—*Reservoir.*—The U.D.C. has applied for a loan of £1,000 for the construction of a reservoir.

FALKIRK.—*Bridge.*—The Stirlingshire County Road Board recommends the construction of a reinforced concrete bridge over the River Carron near Larbert.

FAREHAM.—*Sea Defence Works.*—The R.D.C. has appointed a committee to inquire into a proposed scheme for the construction of 1,400-ft. sea-wall, estimated to cost £14,720.

FORFAR.—*Concrete Bridge.*—A 20-ft. reinforced concrete bridge is to be erected on the main road at Glenisla.

HAVANT.—*Cottages.*—The R.D.C. is to build 80 cottages.

HOLBEACH.—*Houses.*—The U.D.C. is paying £700 for a site for houses.

HONSFORTH.—*Houses.*—The U.D.C. has applied to the M.H. for sanction to erect 100 houses.

HOPPITT.—*Bridge.*—The Essex C.C. is considering a scheme for the erection of a bridge at Hoppitt.

HUNTINGDON.—*Houses.*—The T.C. has decided to erect 50 houses.

LEEDS.—*Road.*—The General Purposes Committee is making provision for a road to link up the Meanwood housing site with Potternewton Lane.

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ENGINEER, fully qualified to take charge of any mining work, also water and sewage contracts. First-class knowledge of concrete and timbering. Box No. 2214, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 20 Dartmouth Street, Westminster, S.W.1.

CLERK OF WORKS, actively employed for many years on public works schemes, water, drainage, and concrete roads, desires re-engagement. Box No. 2215, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 20 Dartmouth Street, Westminster, S.W.1.

CLERK OF WORKS, good knowledge of concrete house and road construction, seeks post. Excellent references. Box No. 2216, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 20 Dartmouth Street, Westminster, S.W.1

LUTON.—*Houses.*—The T.C. has applied to the M.H. for a loan towards the construction of 200 houses.

NOTTS. — *Bridges.* — The C.C. has a scheme on hand for the construction of bridges at Sutton, Crow Park Station, and Muskham. The total estimated cost is £60,000.

POOLE.—*Road.*—The T.C. has submitted plans to the Board of Trade for a proposed road from Green Road to Whitecliff.

PORTDINORWIC. — *Reservoir.* — The Gwyrfai R.D.C. has applied for a loan of £1800 for the construction of a service reservoir at Portdinorwic.

RHAYADER.—*Footbridge.*—The Chamber of Commerce has passed a resolution for the construction of a footbridge over the G.W.R. at Rhayader Station.

ROTHERHAM.—*Houses.*—The T.C. has been recommended to apply for financial assistance towards the erection of 500 houses.

SCARBOROUGH.—*Sea Defence Works.*—The Borough Engineer has submitted a scheme to the Corporation for the construction of 20-ton concrete blocks on the North Side, estimated to cost £45,000.

SEAHAM HARBOUR. — *Houses.* — The U.D.C. proposes to purchase land for a site for 300 houses.

SOUTHAMPTON. — *Houses.* — The Corporation has applied for sanction to erect 300 houses.

STEVENAGE.—*Sewage Works.*—The U.D.C. has accepted the tender of Messrs. L. J. Speight, Ltd., at £9,621, for the construction of sewage disposal works.

TILBURY.—*Concrete Cottages.*—The U.D.C. has accepted the tender of the Triangular Construction Co., Ltd., for the erection of 87 concrete cottages.

TIVERTON.—*Houses.*—The R.D.C. proposes to erect 100 houses.

WAKEFIELD.—*Road.*—The W. Riding C.C. proposes to reconstruct portions of the Wakefield and Austerlands, and Sedburgh and Haws main roads with concrete blocks.

WALLSEND.—*Houses.*—The T.C. is to build 72 houses by direct labour.

WESTWARD HO!—*Sea Defence Works.*—The U.D.C. is appealing for subscriptions towards the cost of sea defence works at Westward Ho! estimated to cost £22,000.

WORCESTER. — *Concrete Houses.* — The City Council proposes to erect 52 concrete houses in Bromwich Road.

Tenders Accepted.

BRIGHTON.—*Reservoir.*—The Corporation has accepted the tender of Messrs. Williams, Tarr & Co., Ltd., Warrington, at £10,940, for the construction of a 1,500,000-gallon reinforced concrete reservoir.

BRIGHTON. — *Roof.* — The Corporation has accepted the tender of Messrs. Williams, Tarr & Co., Ltd., Warrington, at £1,058, for the construction of a reinforced concrete roof at the Shoreham pumping station.

CARNARVON.—*Wall.*—The C.C. has accepted the tender of Messrs. Horton & Co., Colwyn Bay, at £2,855, for the construction of a reinforced concrete wall at Tyddynbadyn.

DROGHEDA. — *Concrete Houses.* — The C.B. has accepted the tender of Mr. John Gagarty, Drogheda, for the construction of 40 concrete houses, at an average cost of £263 1s. 6d. each.

ELGIN. — *Pavements.* — The T.C. has accepted the tender of Mr. A. Falconer, Elgin, for concreting pavements.

GLASGOW.—*Bridge.*—The Corporation has recommended for acceptance the tender of Messrs. Melville, Dundas & Whitson, at £30,525, for the construction of a reinforced concrete bridge over the River Kelvin at Maryhill Road.

GLYNCORRWG. — *Reservoir, etc.* — The U.D.C. has accepted the tender of Mr. John Morgan, Neath, at £8,289 2s. 2d.,

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for the construction of a concrete reservoir, filter beds, etc.

HANLEY.—*Floor.*—The Stoke T.C. has accepted the tender of Messrs. C. Cornes & Son, Hanley, at £245 10*s.*, for the construction of a concrete floor at the destructor works.

HENLEY.—*Concrete Houses.*—The T.C. has accepted a tender of the Calway Construction Company at £26,060 for the construction of 50 concrete houses.

HULL.—*Street Works.*—The Corporation has accepted the tender of Messrs. Quibell & Son, Ltd., Hull, at £225, for the construction of a reinforced concrete slope in place of the steps at Craven Street Bridge.

MACDUFF.—*Reservoir.*—The T.C. has accepted the tender of Messrs. C. MacDonald & Sons, Dyce, at £1,299 10*s.*, for the construction of a concrete reservoir.

PORTSMOUTH.—*Bridge.*—The T.C. has accepted the tender of Mr. J. Croad, Portsea, at £16,865, for the construction of a concrete bridge in place of the existing iron bridge over Portscreek. (This price includes maintenance for twelve months, and the removal of the present bridge.)

PUDSEY.—*Sewer.*—The C.B. has accepted the tender of Messrs. Phineas Drake & Sons, Bradford, for the construction of a mass concrete stormwater sewer.

SMETHWICK.—*Gasholder.*—The T.C. has accepted the tender of the Birmingham Reinforced Concrete Co. for the construction of a reinforced concrete and steel-framed gasholder.

SUNDERLAND.—*Bridges.*—The T.C. has accepted the tender of Messrs. Purdie, Lumsden & Co., Newcastle, for the construction of two steel and concrete bridges, together with retaining walls.

SUTTON COLDFIELD. — *Bridge.* — The T.C. has accepted the tender of the Reinforced Concrete Construction Co., Ltd., Manchester, at £1,031 12*s.* 7*d.*, for the construction of a reinforced concrete bridge over Plants Brook in Penns Lane.

YATTON.—*Bridge.*—The Long Ashton R.D.C. has accepted the tender of Mr. E. Ireland, Bath, at £1,495, for the reconstruction of North End Bridge in reinforced concrete.

New Companies Registered.

Building Block and Slab Co., Ltd. (205637). Registered May 1. Manufacturers of and dealers in concrete blocks, etc. Nominal capital: £100. Directors: P. E. Stenning, Limpsfield, Surrey; Major M. Foyle, 5 Nottingham-terrace, Regent's Park; F. Stocken, 140 Long Acre, W.C.2; G. Heard, Parkside, Buckingham Road, Banstead.

Concrete Mixers, Ltd. (205620) Registered April 30. 27 St. James' Square, S.W.1. Directors: J. Sugden, 2 Cornwall Terrace, N.W.1; N. S. Hedderwick, Santa Rose, Northwood, Middlesex.

Concrocks, Ltd. (205584). Registered April 20. Filsham Waterworks Road, St. Leonards-on-Sea. Manufacturers of concrete blocks, etc. Nominal capital: £2,500. Directors: C. D. Cayley, 3 Park Mansion, St. Leonards-on-Sea; E. G. Tewson, "Kenwood," Terminus Avenue, Bexhill-on-Sea; E. St. Q. Cayley, "Clougha," Radlett, Herts.

Dow's Granolithic, Ltd. (205585). Registered April 29. Manufacturers of granolithic materials, reinforced concrete, etc. Nominal capital: £2,000. Directors: J. C. Dow, 32 Upper Street, Mary's Road, Bearwood, Birmingham; J. K. O'Dowd, 63 Beech Road, Sparkbrook, Birmingham; G. Elvins, Luttrell House, Four Oaks, Warwickshire; F. C. Elvins, 206 Hamstead Road, Handsworth, Birmingham.

E. Stock Portland Cement and Asbestos Products, Ltd. (205621), 176 Upper Thames Street, E.C. Registered April 30. Nominal capital: £1,500. Directors: E. G. Randall, "Aldonca," Lymington Avenue, Leigh-on-Sea; E. C. Creed, Biddenham, Bedford; E. J. Brown; A. Bunyel.

Eastwood's Cement Ltd. (205752). Registered May 6. 47 Belvedere Road, Lambeth. Nominal capital: £150,000. Directors: H. Boot, 7 Victoria Street, S.W.1; F. Alder, Victoria House, Kingsway, Hove; J. MacGregor, 2 Kidbrook Park Road, S.E.

Novocrete and Cement Products Co., Ltd. (205996) Registered May 15. 1–3 Regent Street, S.W.1. Nominal capital £250,000. Directors: Lt.-Col. R. B. Hay, 9 Chester Place, W.2; G. R. Holland, Langley House, Kings Langley; E. C. Holloway, Heather Grange, The Drive, Belmont; H. M. Blenkarne, 9 Chester Place, W.2; G. O. Case, 5 Denmark Terrace, Brighton.

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Trade Notices.

We have received from the United Strip and Bar Mills, Ltd., of The Ickles, Sheffield, a copy of a stock list of mild steel rounds, squares, flats, hexagons, strip, hoop, rivet bars, etc., which they propose to issue fortnightly. The object of this periodic list is to enable customers to know just what the firm has in stock, and obtain quick delivery. We are informed that steel ordered from these lists will be despatched in all cases within 24 hours, and that an order received by 2.30 p.m. will be loaded complete on rail by 6.30 the same evening in time to catch a special shunt each evening from the Ickles works, which has been arranged with the Railway. These facilities should be specially valuable to those requiring rapid delivery of materials to carry out urgent work. In the first list the mild steel rods and bars are quoted in a variety of sizes generally used for reinforced concrete work.

An innovation in the ordinary coloured pencil manufacture—the production of thin leads instead of the large diameter leads hitherto employed—has been made by the American Lead Pencil Co. This change adapts the coloured pencil to a great variety of uses, especially in connection with plans, charts, and diagrams. The colours available are blue, red, green, and yellow, and the composition used for the lead permits sharpening to a fine point and presents the frequent breakages of leads. Samples of the new pencils can be obtained from Alpco Pencils Limited, 173/175 Lower Clapton Road, London, E.5, by readers mentioning this journal.

The results of the tests by Dr. Oscar Faber on rapid-hardening "Ferrocrete" Portland Cement, outlined in our issues for January and February this year, have now been embodied in a booklet by the Cement Marketing Co., Ltd., from whom copies may be obtained on application.

The *Edgar Allen News* for May–June, 1925, issued by Messrs. Edgar Allen & Co., Ltd., of Imperial Steel Works, Sheffield, contains an article on stainless steel and iron and many other articles and notes of interest to those concerned with the manufacture and use of iron and steel.

Messrs. Brunner, Mond and Co., Ltd., have issued a new booklet "P. 84 Silicate of Soda," which gives detailed instructions for the application of this preparation which has been prepared specially for concrete hardening. The method of application is to mix 1 part of the silicate of soda with 4 parts of water and spray or brush the mixture over the surface, which should be reasonably clean, dry and free from grease to ensure adequate penetration. The result is the formation of a dense and hard outer layer, extending to a depth of about ½-in. which is wear-resistant, waterproof, and oilproof. One gallon of silicate of soda mixed with 4 gallons of water is sufficient for the treatment of 40 sq. yds. of surface, and the average cost is stated to be about ½*d*. per sq. yd. for the 3 coats recommended.

We have received from the Harvie Corporation, Ltd., of 11 Queen Victoria Street, E.C.4, a series of leaflets dealing with "Cemenfortis," a liquid for the induration of cement and concrete by chemical combination of the cement and sand. It is claimed that this solution, in addition to making concrete more plastic and easily handled, prevents the disintegration of floors, permits hydration at a temperature of 130° F., and renders concrete waterproof.

Messrs. F. A. Macdonald & Partners, of 135 Wellington Street, Glasgow, have issued a well-produced catalogue of 124 pp. dealing with the "M. & P." System of reinforced concrete construction. The system of reinforcement employed is fully illustrated and described, and many useful calculations and formulæ for reinforced concrete design are given. The second part of the catalogue consists of views and descriptions of a large number of reinforced concrete buildings erected to the designs of the firm, sections being given in many cases; these structures range from monumental buildings to warehouses and coal bunkers, dams, and tanks.

Particulars of a series of tests undertaken in Canada have been forwarded to us by Messrs. Super-Cement, Ltd., of 10 Upper Woburn Place, W.C.1. The object of the investigations was to ascertain the resistance of Super-Cement to alkali, and it is shown that whereas concrete made with other Portland cements disintegrated, specimens made with Super-Cement showed no effect except when in a lean mixture.

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume XX. No. 7 LONDON, JULY, 1925.

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EDITORIAL NOTES.

London Building Acts and Regulations.

MR. TOPHAM FORREST'S Report to the London County Council on his visit to America, which we summarised in our last issue, contains several expressions of opinion on matters with which not all our readers will agree. As regards the thickness of concrete required for foundations, for instance, we think experience in London has shown that except in very special cases 4 in. projection of concrete beyond the lowest course of footings and a thickness of concrete of 9 in. have proved to be safe and sufficient, and under these circumstances we should view with much concern any attempt to make every building subject to the provisions of the 1909 Act, which is very onerous in regard to the time and energy that have to be spent in interviewing District Surveyors and London County Council officials. This may be justified in certain cases, but we think it would be a great mistake to extend this principle and the necessity for this additional supervision to all buildings.

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It is stated in the report that there is little or no demand for radial, or "mushroom," construction in this country, but it should be pointed out that this type of construction has been viewed with distinct disfavour by the London County Council, and we believe that had it been the general impression that the L.C.C. were prepared to consider such construction on its merits many examples of this construction would have been usefully employed.

In connection with the protection of steel by concrete, we agree that excellent protection is obtained by concrete rich in cement and well graded as regards the sand and ballast, but we strongly deprecate specifying the volume of aggregate, including sand, instead of specifying so much gravel plus sand. It completely changes the notation which has now become commonly understood throughout the industry, and in itself carries no corresponding advantage with it, as the proportion can be quite well specified under the old system to give any particular ratio between the two that may be required. We do not agree that it gives greater freedom to the designer, and we think the only result is to produce a fresh possible cause of chaos into an industry already suffering from constant changes in the standard notation and in the method in which the requirements are stated. We quite agree that there should be enough sand to fill the interstices between the gravel, but this can be easily obtained under the present methods of specifying proportions.

In regard to the measurement of cement, we are entirely in agreement, and think it is an improvement to measure cement by weight and not by volume; this has already been advocated by many English engineers for several years, however, and it must not be thought that America is entitled to the credit of having introduced this innovation.

We quite agree that the quantity of water is very important, although it is certainly an overstatement to say that a difference of 1 per cent. of water may mean a difference of 1,000 lbs. per sq. in. in the crushing strength of concrete. We agree that concrete which can be used in a chute is wetter than is consistent with the best concrete.

We notice that it is proposed to increase the stress in the reinforcement from 16,000 to 20,000 lbs. when secondary stresses are taken into account. With this we entirely agree, and it is a coincidence that the same proposal has for three years been adopted by the Science Committee of the Institution of Structural Engineers, which has the L.C.C. regulations under consideration.

With the suggestions in regard to hollow reinforced concrete floors we generally agree, though we think that the upper portion of the blocks could have been taken into account as resisting the compressive stress and we believe the tests indicate that this does in fact occur even when no special steps are taken to get more than the usual connection between the ends of the blocks.

Before the recommendations and suggestions contained in the report are embodied in definite Regulations or Building Acts we hope the points enumerated above will receive consideration.

Tidying up London.

WHILST so much is heard from the "Brighter London" enthusiasts, and a discussion which seems to be out of all proportion to its importance rages around

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the Hudson Memorial in Hyde Park, cannot something be done to lessen the hideousness of one of the worst eyesores in London? We refer to the Science Museum at South Kensington. Erected on a splendid site in the midst of the University quarter, the building devoted to science and now housing the War Museum should at least be as good to look upon as the very fine buildings with which it is surrounded. But what do we find? Although the structural work was completed five years ago, the elevation has never been finished. A temporary concrete wall, and a very ugly one at that, was put in position to fill in the frontage pending the execution of the façade in Portland stone, and there it remains to this day. One can understand that five years ago, when so much leeway in building work had to be made good, there was some reason for not proceeding with the handsome stone frontage which had been designed for this building, but does the same reason hold good to-day? Surely the completion of an important building such as this should take precedence over much of the Government work we see going on at present.

Our interest in this building is a very real one, for with the exception of the elevations it is a complete reinforced concrete building which we looked forward to as being, when finished, an excellent example of what can be accomplished with this material. At a time, however, when it should have been finished, we hear it referred to as a "concrete monstrosity" by the press and by people who do not realise that it is unfinished. The ugly temporary wall is looked upon as an example of a concrete façade, and those who see it without knowing the position may be excused for retaining a bad impression of concrete as an architectural material for the rest of their lives.

If it is still impossible to secure Portland stone at a figure which the Government is prepared to pay, would not this building, already a complete reinforced concrete structure, be a suitable one for finishing in cast concrete? One has to go no farther than the City of London to see what can be done with cast stone, for it is being used by some of the leading architects on many of the most important new buildings. We would suggest that a competition among architects be held for a new design for the façade of the Science Museum, to be carried out entirely in pre-cast reinforced concrete in harmony with the rest of the building. Such a material would be considerably cheaper than Portland stone, and would not call for the employment of stonemasons. We commend the matter to the consideration of the London Society and the Architectural societies, who might well turn their attention to these smaller things that badly need seeing to rather than spend so much energy on such proposals as a new Charing Cross Bridge which cannot possibly be carried to fruition within the next few years.

The Effect of Storage upon Cement.

THE subject of the effect of storage upon cement is one that interests several classes of buyers and users, including small merchants who buy large parcels in order to obtain low transport rates and are only able to dispose of the full quantity after several months, contractors who by reason of bad weather or other causes are unexpectedly hindered in their work and the consequent use of the cement, and foreign and Colonial buyers who do not receive the cement until several weeks after despatch. The subject is becoming increasingly important because

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the finely-ground cement that is now usual is more liable to depreciate by absorption of moisture from the atmosphere than was the coarsely-ground product of twenty years ago. This latter cement was indeed frequently improved and made sounder by storage and air slaking, whereas the well-manufactured cement of to-day is in its best condition immediately on leaving the factory.

The report of an investigation upon the subject ("Effect of Storage of Cement," by Duff A. Abrams. Bulletin 6 of the Structural Materials Research Laboratory, Chicago. Second edition, April, 1924) is therefore welcome and repays study. This investigation has been carried out very thoroughly and includes tests at several periods up to two years of cement stored for various periods up to five years. The report shows how the compressive strength of cement changes after different periods of storage under certain conditions. When stored in sacks in a shed such as would be used by a contractor, the strength (as averaged at all ages) decreased as follows, the figures showing the percentage compressive strength (of a mixture of 1 part of cement to 5 parts of aggregate) as compared with the cement before storage.

Cement as received	100	per cent.
,, after 3 months' storage	80	,,
,, ,, 6 ,, ,, ...	72	,,
,, ,, 1 year's ,, ...	60	,,
,, ,, 2 years' ,, ...	46	,,

Thus the cement after two years' storage had less than half the strength of the original cement. It is important to observe, however, that the concrete made with a cement deteriorated by storage improves with age. For example, with cement stored in sacks for 3 months, the 7-days' test of concrete (1 to 5) was 73 per cent. of the amount shown by the freshly manufactured cement, while the 28 days' test was 75 per cent. and the other tests from 6 months to 2 years were 84 per cent. of the fresh cement tests. Hence a cement damaged by storage is slower in hardening than a fresh cement. The sacks in which the cement was stored were of cloth as used in the United States, but storage in paper sacks did not give appreciably different results. Much depends upon the conditions of storage. The cement in sacks could be stored in a heated room at a uniform temperature for twice as long as in a contractor's shed for the same amount of deterioration, and cement can be stored in bulk in large bins at cement works for one and a half years with practically no deterioration.

The conclusions to be drawn from this investigation are that as far as possible cement should not be kept for long periods in sacks, and where prolonged storage is unavoidable the sacks should be emptied and the cement stored in a compact bulk.

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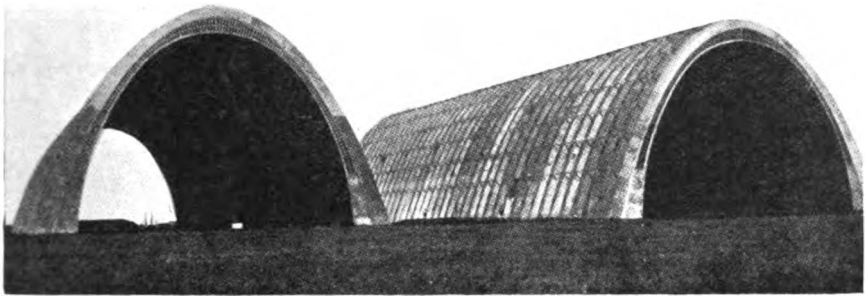


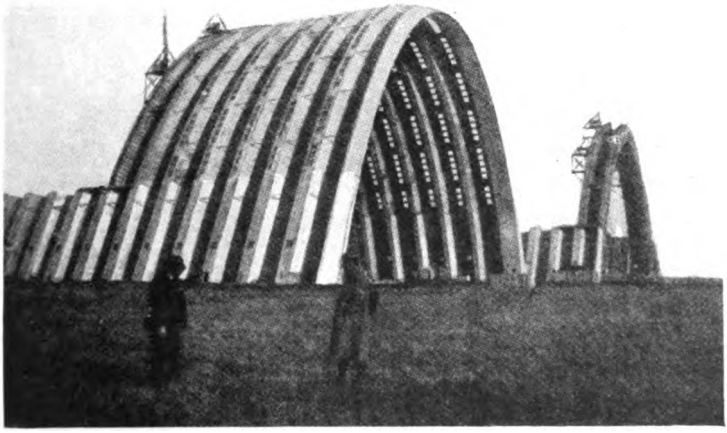
Reinforced Concrete Airship Sheds at Orly, France.

Reinforced Concrete Airship Sheds.

THE airship hangars illustrated herewith have been erected for the French Government at Orly, near Juvisy, from the designs of M. Freyssinet, which were selected in competition. The two hangars each measure 1,000 feet long by 300 feet wide by 196 feet high.

The foundations of the arches consist of slabs of reinforced concrete 3 feet thick in trenches 6 ft. 6 in. deep by 25 ft. wide, extending along the whole length of the hangars, and on this was built a base extending about 3 ft. above ground level and conforming to the corrugated outline of the inner and outer faces of the structure. The dimensions of these segments are given in the section.

As will be seen from the illustrations, the method of construction was first to erect arched supports for the forms, which were placed in position by derricks, mounted on trucks with concrete platforms travelling on tracks throughout the length of the structure. These arched form supports are of timber, of the same size as one segment, and they were moved along after the concrete had been poured in the forms to support the same forms



View showing Inner and Outer Surfaces of Hangar and Windows.

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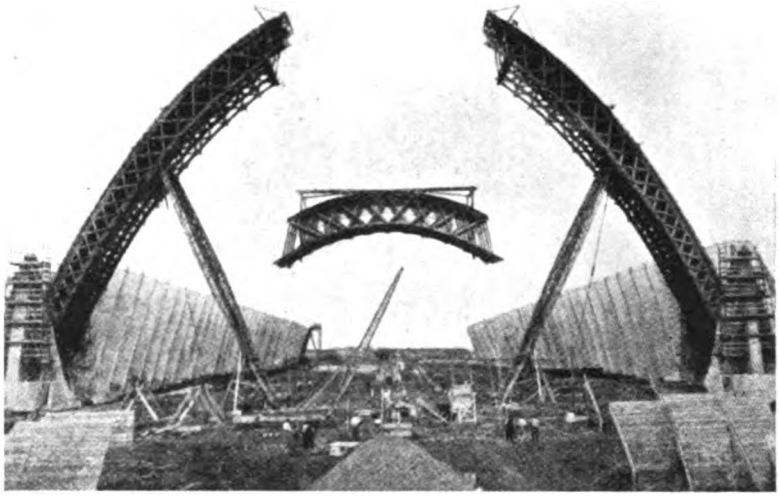
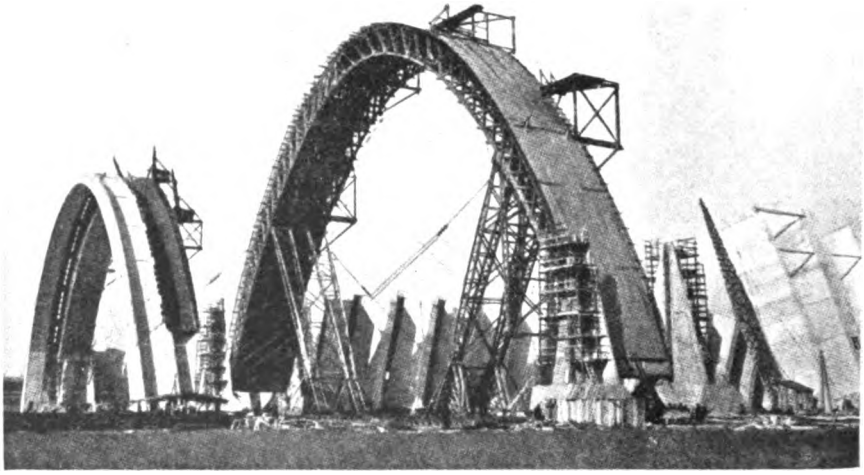


Erecting Form-Supports.



Form Supports in Position. On the left one segment has been completed and the Form-Support and Forms moved along for the next segment.

Reinforced Concrete Airship Hangars in France.

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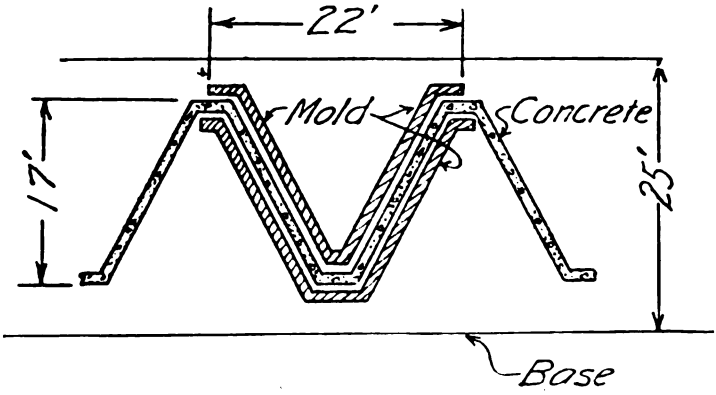

for pouring the next segment. These form supports were made in three sections, the middle or top section acting as a key and placed in position by winches on the top ends of the side sections. The form supports were thus self-supporting arches, the lower ends of which were mounted on a reinforced girder which could be lowered about 6 ft. after the concrete was placed in order to release them for moving along to the next segment.

The formwork, in sections, was elevated into position by the derricks on the supporting arches. The time taken to pour each segment was seven days, each segment being 35 ft. wide and 15 in. thick. At the crown of the arch the concrete is 3½ in. thick at the sides of the segments and 8 in. thick at the top and bottom, while at the lower parts of the segments the concrete increases to 13½ in. in thickness at the top and bottom of the segments. We are indebted to the "American Architect" for illustrations and the above notes.

Renewing a Bridge Deck with Reinforced Concrete "Planks."

In order to minimise interruption of traffic while the roadway of a bridge in Ontario was being replaced, it was decided to re-pave with reinforced concrete planks in place of the existing wooden deck. The roadway of the bridge was 18 ft. wide, and the "planks" were cast in lengths of 11 ft. and 7 ft., in each case the section being 12 in. wide by 4 in. deep. The reinforcement consisted of three ⅜-in. round longitudinal rods and ¼-in. transverse rods at 4-in. centres. The edges were grooved and tongued so that the "planks" fitted into each other. Altogether 520 of these "planks" were used, the joints being grouted. The units were made near the site of the bridge and cured for 28 days before the work of replacement was started. The old timber was removed and the concrete units placed in position at the rate of 80 or 90 a day, and the work was so arranged that traffic was only kept off the bridge for eight out of each twenty-four hours while it was in progress. Doubts were expressed as to the ability of the steel trusses to carry the extra weight of the concrete decking, but it was found that the new floor was only 30 per cent. heavier than the existing rain-soaked timber. By this method not only has a permanent deck been obtained, but as the floor is supported direct on the steelwork, the method adopted avoided the expensive formwork which would have been necessary had the concrete been poured *in situ*.

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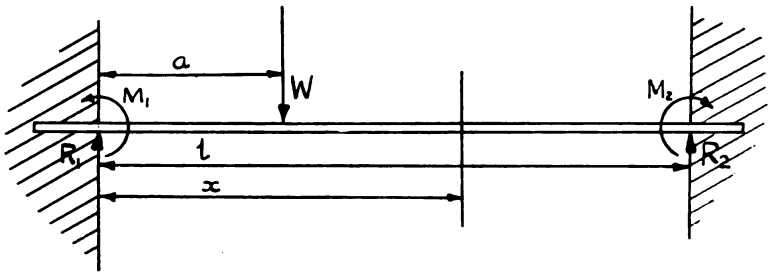

FIG. 1.

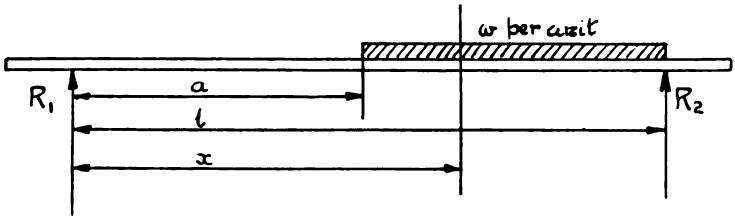

FIG 2.

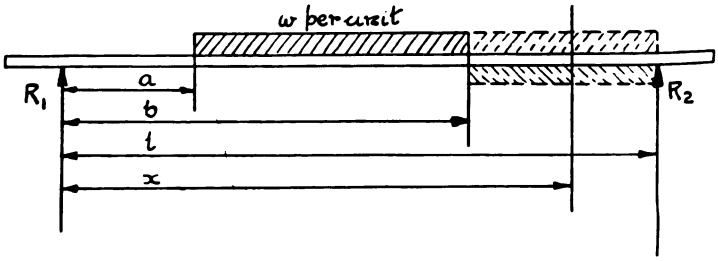

FIG. 3.

THE ELASTIC EQUATION FOR BEAMS. (*See p.* 367.)

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THE ELASTIC EQUATION FOR BEAMS.

W. D. WOMERSLEY, M.A., B.Sc., A.M.Inst.C.E.

THE powerful single equation method for the determination of the bending moments, reactions, etc., in beams due to any kind of loading, fixing, and sinkage conditions which has been in use in the Engineering Schools of Cambridge University for the past twenty years or more does not even yet seem to be very generally known, and as its application is so universal and the simplification produced so enormous no apology will be needed for re-expounding it here.

The general method of applying the elastic equation to beams with unsymmetrical loading and discontinuities seems to be to write down a separate equation for each section of the beam between the discontinuities and to interconnect the various quantities through the slope and deflection at the discontinuous points. For example, consider an encastré beam with an unsymmetrical point load as shown in *Fig.* 1. Then measuring (x) from the left end of the beam and (y) positive in the downward direction below the unbent centre line, $EI\frac{d^2y}{dx^2}$ will be negative and clockwise bending moments will be positive. Thus for the part of the beam where $x < a$

$$-EI\frac{d^2y}{dx^2} = R_1x - M_1 \quad . \quad . \quad . \quad . \quad . \quad (1)$$

and for $x > a$

$$-EI\frac{d^2y}{dx^2} = R_1x - M_1 - W(x - a) \quad . \quad . \quad . \quad (2)$$

Now if these two equations are integrated separately and all the constant parts placed into a composite constant, as obtained, two sets of apparently different equations will ensue. On the other hand, if the integrations are carried out in a certain definite manner the integration constants in the two sets of equations will be found to have the same value and thus the two equations can be merged into one with slight restrictions. Thus from equation (1)

$$-EI\frac{dy}{dx} = \frac{R_1x^2}{2} - M_1x + A \quad . \quad . \quad . \quad . \quad (3)$$

$$-EIy = \frac{R_1x^3}{6} - \frac{M_1x^2}{2} + Ax + B \quad . \quad . \quad . \quad (4)$$

and from equation (2)

$$-EI\frac{dy}{dx} = \frac{R^1x^2}{2} - M_1x - \frac{W}{2}(x - a)^2 + C \quad . \quad . \quad . \quad (5)$$

$$-EIy = \frac{R_1x^3}{6} - \frac{M_1x^2}{2} - \frac{W}{6}(x - a)^3 + Cx + D \quad . \quad . \quad (6)$$

in which it is seen that $(x - a)$ is integrated as a whole which will cause the constant C to be different by the quantity $\frac{Wa^2}{2}$ from the one obtained in the usual way. Now, by putting in the value $x = a$ in equations (3) and (5) it will

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be seen that the constants A and C have the same value and similarly by putting in the value $x = a$ in equations (4) and (6) it will be evident that the constants B and D have the same value. Thus it follows that the equation (2) can be applied to the whole beam with the one condition attached, and that is that the term $\{W(x - a)\}$ is left out if $x < a$; or, in other words, if the part within the brackets { } becomes negative. This means that only two constants are to be found instead of four, and only one equation is to be operated on instead of two or more dependent on the type of loading on the beam. Thus, recapitulating or the given case :—

$$-EI\frac{d^2y}{da^2} = R_1x - M_1 - \{W(x - a)\} \quad . \quad . \quad . \quad (7)$$

$$-EI\frac{dy}{dx} = \frac{R_1x^2}{2} - M_1x - \left\{\frac{W}{2}(x - a)^2\right\} + A \quad . \quad . \quad (8)$$

$$-EIy = \frac{R_1x^3}{6} - \frac{M_1x^2}{2} - \left\{\frac{W}{6}(x - a)^3\right\} + Ax + B \quad . \quad . \quad (9)$$

If the end conditions are such that when

$$x = 0 \text{ and } l \qquad y \text{ and } \frac{dy}{dx} = 0$$

then $A = B = 0$

and from (8)

$$0 = \frac{R_1l^2}{2} - M_1l - \frac{W}{2}(l - a)^2$$

and from (9)

$$0 = \frac{R_1l^3}{6} - \frac{M_1l^2}{2} - \frac{W}{6}(l - a)^3$$

from which

$$R_1 = \frac{W(l - a)^2\ (l + 2a)}{l^3}$$

$$M_1 = \frac{Wa(l - a)^2}{l^2}$$

R_2 and M_2 can then be found at once by the usual application of statics.

In the case of a uniformly distributed load extending from the end of the beam similar methods are applied. The beam should be arranged so that the load runs from the right hand end, otherwise the various signs will have to be changed and it is much simpler to keep the method quite mechanical. Thus for the case shown in *Fig.* 2

$$-EI\frac{d^2y}{dx^2} = R_1x - \left\{\frac{w(x - a)^2}{2}\right\}$$

$$-EI\frac{dy}{dx} = \frac{R_1x^2}{2} - \left\{\frac{w(x - a)^3}{6}\right\} + A$$

$$-EIy = \frac{R_1x^3}{6} - \left\{\frac{w(x - a)^4}{24}\right\} + Ax + B$$

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If the conditions to be applied are that $y = 0$ when $x = 0$ and l and that $-EI\frac{d^2y}{dx^2} = 0$ when $x = l$ then $B = 0$

and
$$0 = \frac{R_1 l^3}{6} - \frac{w(l-a)^4}{24} + Al$$
$$0 = R_1 l - \frac{w(l-a)^2}{2}$$

from which equations
$$R_1 = \frac{w(l-a)^2}{2l}$$
$$A = \frac{w(l-a)^2(a^2 - 2al - l^2)}{24l}$$

and the remainder of the solution follows in the usual way.

Two cases require special consideration.

(I) A uniformly distributed load which does not run to the end of the beam, as in *Fig.* 3.

In this case the load must be run to the right hand end of the beam and an equal upward load applied under the extra piece. The extra piece has to be added because a single general equation could not be written down. The single equation now is :—
$$-EI\frac{d^2y}{dx^2} = R_1 x - \left\{\frac{w(x-a)^2}{2}\right\} + \left\{\frac{w(x-b)^2}{2}\right\}$$

and the solution is carried out exactly as before.

(II) A couple applied to some intermediate point on the beam. The couple is split up into two very large forces W at a very small distance apart ($2b$) and such that if the couple applied is C then $C = 2bW$, but b^2W and b^3W etc. will be small quantities of the first and second orders in comparison. An example is shown in *Fig.* 4.

The equation is :—
$$-EI\frac{d^2y}{dx^2} = R_1 x - M_1 - \{W(x-(a-b))\} + \{W(x-(a+b))\}$$

Whence
$$-EI\frac{dy}{dx} = \frac{R_1 x^2}{2} - M_1 x - \left\{\frac{W}{2}(x-a+b)^2\right\} + \left\{\frac{W}{2}(x-a-b)^2\right\} + A$$
$$-EIy = \frac{R_1 x^3}{6} - \frac{M_1 x^2}{2} - \left\{\frac{W}{6}(x-a+b)^3\right\} + \left\{\frac{W}{6}(x-a-b)^3\right\} + Ax + B$$

The end conditions give that

When $x = 0$ $\quad y$ and $\frac{dy}{dx} = 0$

Whence $A = B = 0$

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When $x = l$ $\qquad y$ and $\frac{dy}{dx} = 0$

Thus

$$0 = \frac{R_1 l^2}{2} - M_1 l - \frac{W}{2}(l - a + b)^2 + \frac{W}{2}(l - a - b)^2$$

$$0 = \frac{R_1 l^3}{6} - M_1 \frac{l^2}{2} - \frac{W}{6}(l - a + b)^3 + \frac{W}{6}(l - a - b)^3$$

Now, considering the third and fourth terms of these two equations and leaving out expressions involving Wb^2 and Wb^3 it will be seen that the equations become :—

$$0 = \frac{R_1 l^2}{2} - M_1 l - 2Wb(l - a) = \frac{R_1 l^2}{2} - M_1 l - (l - a)C$$

and

$$0 = \frac{R_1 l^3}{6} - M_1 \frac{l^2}{2} - Wb(l - a)^2 = \frac{R_1 l^3}{6} - M_1 \frac{l^2}{2} - \frac{(l - a)^2}{2} C$$

from which R_1 and M_1 can be at once obtained and the values of R_2 and M_2 will follow from statics.

The method is just as easily applicable to beams of multiple span and can be used in place of the theorem of three moments, with the added advantage that no formulæ or centroid positions need be remembered. For example, consider a beam of, say, three unequal spans with any type of loading, as indicated in *Fig.* 5. The details of the equation may be written down quite mechanically if it is arranged always to work the same way, that is, measure (x) from the left

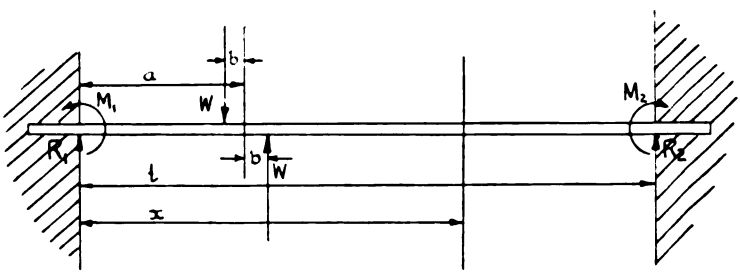

FIG 4.

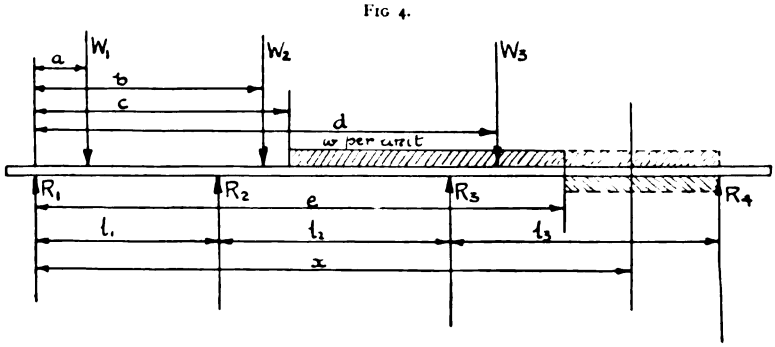

FIG. 5.

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hand end and deflections positive below the centre line of the beam, $\frac{EI d^2y}{dx^2}$ is then negative and clockwise bending moments are positive. For the purpose of writing down the equation (x) must be taken as being beyond the last point load and/or into the last distributed load.

The equation for the beam of *Fig.* 5 is as below, the terms being written down from the left end of the beam :—

$$-EI\frac{d^2y}{dx^2} = R_1x - \{W_1(x-a)\} + \{R_2(x-l_1)\} - \{W_2(x-b)\}$$

$$-\left\{\frac{w}{2}(x-c)^2\right\} + \{R_3(x-(l_1+l_2))\} - \{W_2(x-d)\} + \left\{\frac{w}{2}(x-e)^2\right\}.$$

This equation, on integration, will give five unknowns, R_1, R_2, R_3 and the two constants of integration A and B. Five conditions are given by the deflections at the four supports and the bending moment being zero at the right hand end of the beam. The work involved by an example of this kind is considerable, but is probably not so great as would be required by the Theorem of Three Moments, as no centroids of irregular areas have to be determined. For symmetrical cases the latter theorem would, of course, be much simpler. For those students who care for arithmetical gymnastic exercises the solution of the above example is as under, assuming all the supports to remain at the same level and putting in numerical values.

Let :—

$l_1=20$ ft., $l_2=40$ ft., $l_3=30$ ft., $a=5$ ft., $b=27$ ft., $c=30$ ft., $d=62$ ft., $e=69$ ft., $W_1=10$ tons, $W_2=20$ tons, $W_3=15$ tons, $w=1$ ton per foot.

Then :—

$R_1 = 0{\cdot}22$ ton. $R_2 = 38{\cdot}22$ tons. $R_3 = 47{\cdot}24$ tons. $R_4 = -1{\cdot}68$ tons

For those who are not conversant with the method a little practice will soon give confidence and the terrors usually associated with the elastic equation will vanish. The results of a general expression, when obtained, may always be readily checked by choosing the arbitrary dimensions so as to make the beam symmetrical.

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Reinforced Concrete in Czecho-slovakia.

THE creation of the republic of Czecho-slovakia resulted in the Austrian regulations for reinforced concrete applying in a portion of the new republic and the more stringent Hungarian regulations in other parts. To avoid this complication a new set of regulations has been drafted for the whole State. These rules are the subject of a recent article in *Le Constructeur de Ciment Armé*, from which the following summary is taken.

In ordinary works, provided expansion joints occur at distances of 100 to 130 ft., no account is taken of the stresses caused by temperature variation. In certain cases these distances are reduced in value. In rigid frames and arches a temperature range of $\pm$ 12° C. must be allowed for.

For continuous beams, free at the supports, the possible sinking of the supports must be taken into account. If the beams are not free on the supports the fixation moments must be allowed for, and in the case of continuous slab monolithic with tee-beams and with uniform loading of less than 200 lbs. per sq. ft., the maximum moments may be taken as $+\frac{ql^2}{16}$ on an intermediate span, and at the end support but one, $-\frac{ql^2}{10}$, where l = distance between the ribs.

In the case of point loads, 1·50 times the moment on a single span fixed at both ends is to be allowed as the maximum moment between the supports of an intermediate span. Thus for a point load at the centre of a span, $1{\cdot}5 \times \frac{Pl}{8} = \frac{3}{16}Pl$ is to be used. The fixation moment, however, is to have its real value, i.e. $-\frac{1}{8}Pl$. These values will be increased by 25 per cent. for end spans.

When the spans are very unequal the smallest positive moment is to be taken at not less than one-half the greatest moment produced on a freely supported beam under the same loading.

Rectangular slabs continuous over, partially fixed by, or freely supported at, four edges may be considered as two imaginary slabs of spans a and b. The total load being Q, the load on span a will be

$$Q_a = \frac{b^3}{b^3 + 2{\cdot}3a^3}Q \text{ and on span } b$$

$$Q_b = \frac{a^3}{a^3 + 2{\cdot}3\,b^3}Q, \text{ provided that}$$

$a \leqslant 1{\cdot}5$

If $b = a$, $Q_a = Q_b = 0{\cdot}303Q$ and $Q_a + Q_b = 0{\cdot}61\,Q$

If $b = 1{\cdot}5a$, $Q_a = 0{\cdot}595Q$ and $Q_b = 0{\cdot}114$

$$\therefore Q_a + Q_b = 0{\cdot}71\,Q.$$

Consequently the sum $(Q_a + Q_b)$ of the loads lies between $0{\cdot}71\,Q$ and $0{\cdot}61\,Q$, whereas in other regulations (e.g. the German) it is always equal to Q.

The width of slab assumed as flange of a tee-beam is taken as 20 times the slab thickness.

Compression members more than 18 diameters in length must be designed to resist buckling; when $\frac{l}{d}$ is $>$ 20 and P is the axial load a moment of $\frac{Pl}{200}$ is to be provided for.

When the columns are not connected to the beam in such a way as to form a rigid frame they must nevertheless be designed to take one-third of the fixing moment which would occur in a completely rigid structure. In truly rigid frames the exact calculations must be made and suitable reinforcement used.

For stress determination $\frac{Es}{Ec}$ is taken as 15, and as 10 for calculation of deformations. Tension in concrete is neglected except in the case of combined bending and axial loading.

Compression members axially loaded and not reinforced with spirals are deemed to have an effective section,

$$F_i = F + 15f + \left(18 - \frac{10v}{d}\right)f_s, \text{ where}$$

F = sectional area of the concrete,
f = sectional area of longitudinal bars,
f_s = sectional area of an imaginary longitudinal having, per unit length, the same volume as the hoops,
v = hoop spacing
d = least diameter of the column.

With ordinary hooping the last term is negligible, and with $F_i > 18F$ one must use the value $F^i = 18F$.—W. S. G.

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Reconstruction of the Mining District of Northern France.

By P. Calfas, Ing.E.C.P.

SYSTEMATICALLY destroyed with the object of ruining the industry, the coalfields of Northern France undoubtedly suffered more than any other industry during the occupation. As soon as hostilities came to an end, however, the work of reconstruction was commenced, and has now been practically completed on most of the coalfields in this district. Reinforced concrete has entered largely into this work, and some of the most important structures carried out in this material are described in the following notes.

Lens and Meurchin.

At Lens and Meurchin four new shafts, with all the accessory buildings carried out in reinforced concrete on the "Hennebique" system, have been completed, each shaft having grouped around it a power station, office building, transformer station, and other usual pit-head buildings.

Dourges.

At Dourges the Mulot mine has been provided with a complete installation of surface works, all in reinforced concrete. The building at the mouth of the shaft (*Fig.* 1) measures 73 ft. 6 in. by 20 ft., and is 53 ft. high, covered with an arched roof. The coal is received on the second floor, while the first floor is used for the examination of safety lamps, and the ground floor for sluices. The structure supporting the winding apparatus consists of four columns 109 ft. high and two stays 128 ft. long. The load supported by the pulleys, when the winding apparatus is fully loaded, is 12 tons.

Fig. 1.—Reinforced Concrete Pit Head Buildings at Dourges.

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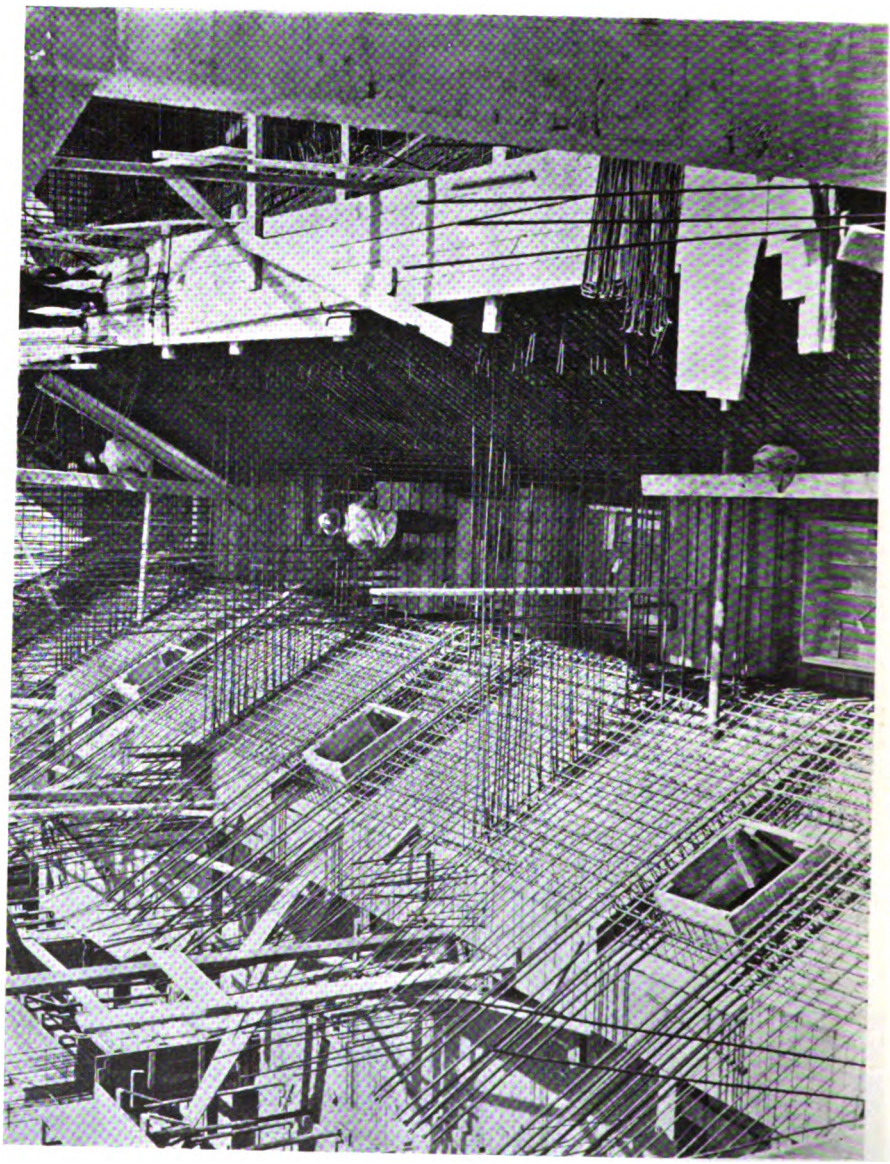

Fig. 4.—Reinforcement of Wall at Aniche Colliery.

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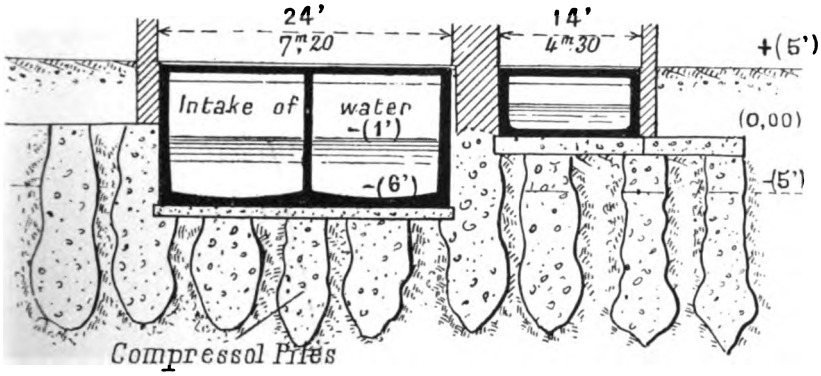

Fig. 2.—Piling at Aniche Power Station.

Aniche.

A 50,000 kilowatt power station and a coal-washing plant have been built by the Aniche Mining Co. The power station, covering an area of 213 ft. by 328 ft., is on marshy soil, and is carried on 974 reinforced concrete piles of the "Compressol" type. On account of the nature of the soil the channels for the intake of water are also supported on "Compressol" piles (*Fig.* 2), some 435 in number, into which was rammed a volume of 12,425 cu. yds. of a mixture of clay and broken brick. This brought about a change in the nature of the soil, which instead of being boggy became clayey and sufficiently firm to carry the water channels.

The new coal-washing plant (*Fig.* 3) covers an area of 361 ft. by 131 ft., and is built in reinforced concrete to a height of 42 ft. 6 in. above ground level. The reinforced concrete pillars are 3 ft. by 3 ft. in section at the base, filled in with a reinforced concrete wall 4 in. thick. The floors throughout are of reinforced concrete. On each of the four sides of the building are sheds with reinforced concrete roof-trusses of 69 ft. span and a cover of 2 in. *Fig.* 4 shows the reinforcement of a wall of a storage building at the same colliery.

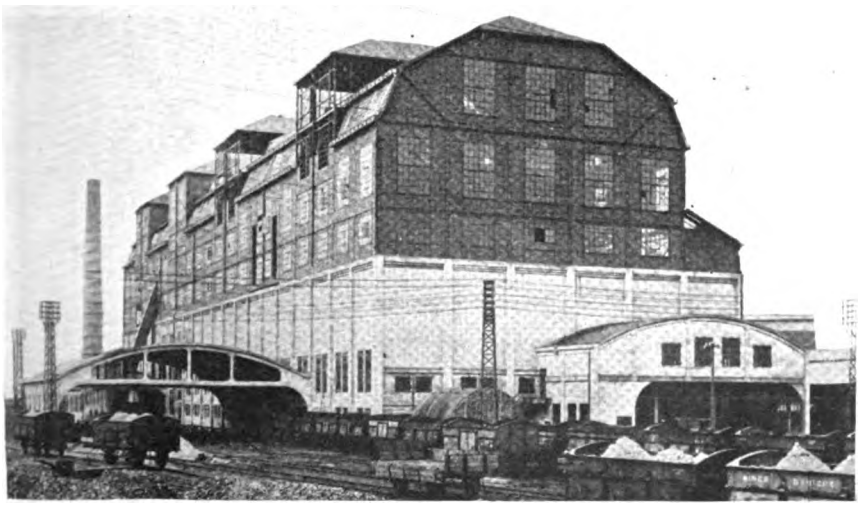

Fig. 3.—Coal-Washing Plant at Aniche.

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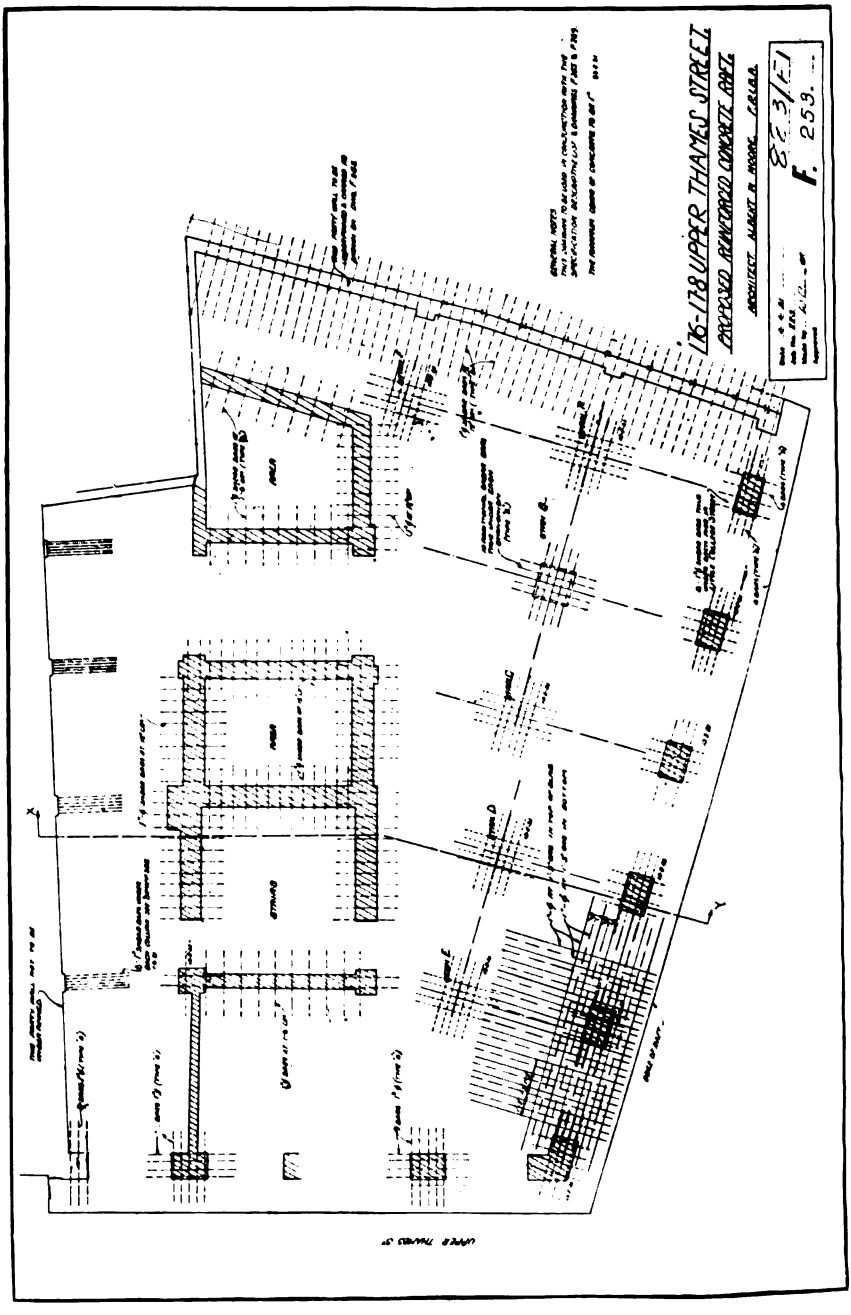

Fig. 6.—Reinforced Concrete Raft in Upper Thames Street, London. (*See p.* 377.)

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Modern Methods of Construction.

THE following is a continuation of a paper entitled, "Modern Methods of Construction," read recently by Dr. Oscar Faber, O.B.E., D.Sc., before the Royal Institute of British Architects:—

In the case of a large warehouse of six stories in Lower Thames Street, for which Mr. Albert Moore, F.R.I.B.A., was the architect, it was found on examining the foundation soil that it consisted of black river mud, and a trial hole indicated that it would be necessary to take the foundations down a matter of 12 ft. below the basement floor and into water before a satisfactory soil in the ordinary sense would be reached.

After some consideration it was decided to float the building on a concrete raft on the black mud at the higher level. It was found that the pressure per sq. ft. under this raft would be of the order of 1,700 lbs. per sq. ft., which it was considered would be safe for the mud in question, though he would not have dared to use it under isolated foundations. *Figs.* 6 and 7 show details of this raft. The builder in question was so alarmed at the instructions to construct this raft on the black mud that he caused his solicitor to write letters of protest to the architect and to disclaim all responsibility for all the terrible things he was certain would result. He followed this up by having levels taken at each stanchion every week. During the construction of the building he continued this, which was no doubt a very wise precaution from his point of view under the circumstances, but about a year after the completion he came to express his amazement at the fact that there had not been $\frac{1}{16}$-in. movement that he had been able to detect with his levelling. A very large saving to the client resulted from this method of dealing with the problem, and I think most of us would have hesitated to found a building on this mud in any circumstances.

A specially interesting raft was used in the Harrow Music School. This structure, which was some 50 years old, was built on the side of a hill at Harrow on yellow clay, which in Harrow is particularly treacherous. The building was originally built on brick piers averaging 14 ft. to 16 ft. below ground floor, and spread on concrete piers averaging perhaps 5 ft. square. A year or two ago settlements in the building were so pronounced as to render it dangerous, and the settlements were proceeding. Some repair work had been done a few years previously, but had not stopped the settlement. Calculations of the bearing pressure under the piers showed that these pressures, though only amounting to about $1\frac{3}{8}$ tons per sq. ft., were more than the clay would resist without settlement, in spite of the depth of the piers. Underpinning would have been a very hazardous and expensive operation, and I recommended the construction of a concrete raft only 8 in. thick to be constructed as a basement floor and chased into the existing piers so that no further settlement could take place without taking the raft with it (see *Fig.* 8). This raft had to be constructed at various levels owing to the slope of the ground, and was an inexpensive operation; no further settlement has taken place. The effect of the raft was to reduce the pressure down to only 1,100 lbs. per sq. ft., which practically any soil will resist without settlement.

Books Received.

"Temperaturschwankungen und Temperaturbewegungen von Beton- und Steinbrücken." By Dr.-Ing. Friedrich Vogt. (Berlin: Wilhelm Ernst & Sohn. Price 5 Marks.)

"Die Eisenkonstruktionen des Hochbaues." By Lauenstein-Bastine. (Leipzig: Alfred Kroner Verlag. Price, Vol. I, 5 Marks; Vol. II, 5 Marks.)

"Schwindspannungen in Trägern aus Eisenbeton." By Leopold Herzka. (Leipzig: Alfred Kroner Verlag. Price 5 Marks 50.)

"Die Festigkeitslehre." By R. Lauenstein. (Leipzig: Alfred Kroner Verlag. Price 5 Marks.)

"Die Graphische Statik." By Lauenstein-Bastine. (Leipzig: Alfred Kroner Verlag. Price 5 Marks.)

"Der Eisenbeton." By R. Saliger. (Leipzig: Alfred Kroner Verlag. Price 24 Marks.)

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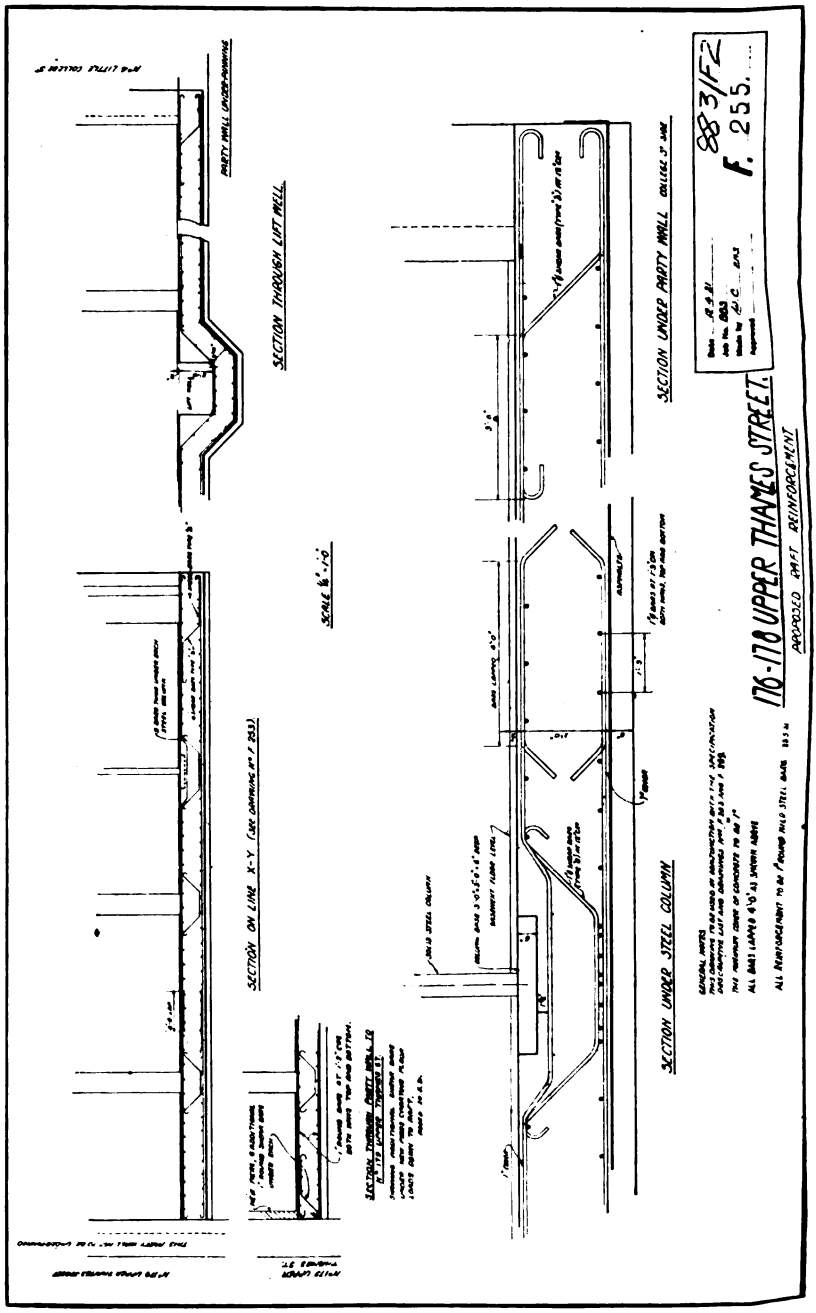

Fig. 7.—Reinforced Concrete Raft in Upper Thames Street, London. (See p. 377.)

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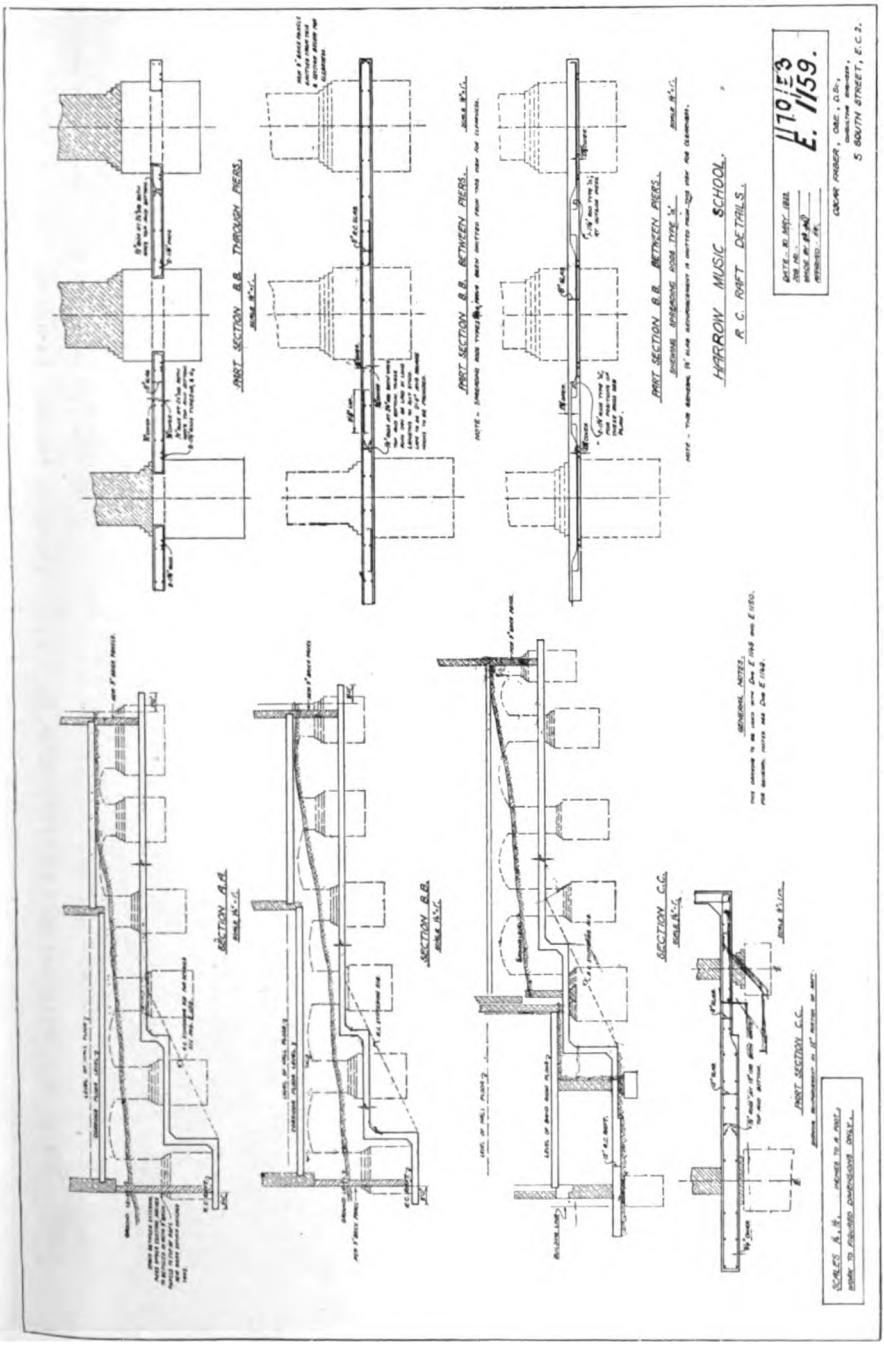

Fig. 8.—Reinforced Concrete Raft at Harrow School. (*See p.* 377.)

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Floating Plant at Ham.

Loading Depot near Ham Common.

A Modern Aggregate Plant. (*See p.* 381.)

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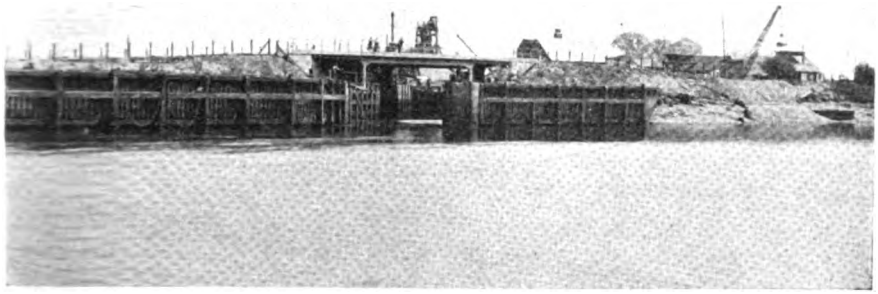

Entrance to Lock at Ham.

A Modern Aggregate Plant.

It is one of the advantages of a new industry that it starts unfettered by old customs and antiquated ideas which are so often seen to hamper industries of older standing, but although reinforced concrete construction is a comparatively new industry some of the ideas which have grown up with the use of plain concrete through the centuries are found still to cling to the modern reinforced concrete. One of the old ideas was that any kind of aggregate would do for mixing with cement to make concrete, and unfortunately we still sometimes see dirty material of a nondescript character used in constructional work where if the strength for which the concrete has been designed is to be reached only the best materials should be used. Cement and steel are both governed by standard specifications, and can be ordered with the assurance that they will be exactly to requirements, but the other ingredient, and the material which forms the bulk of the finished concrete, is often governed by no specification, and if not obtained from a reliable source or carefully watched as it is delivered will detract to a large extent from the ultimate strength desired. There is, however, an increasing realisation of the importance of using only thoroughly clean and graded aggregates for reinforced concrete work, and many of the businesses engaged in supplying these materials have marched with the times and installed plant which ensures that they are of first-class quality.

Typical examples of up-to-date aggregates plants are those of the Ham River Grit Company, at Ham, and its associated concern, Concrete Aggregates, Ltd., at Chiswick. At Ham, near Richmond, the works are situated at the side of the river, and by building a lock at the towing-path and allowing water to run from the river into the area which is being excavated a considerable lake has been formed. The main plant here consists of two pontoons on which are erected the grading screens and crushers, and from these floating plants the graded material is delivered into barges moored alongside and brought direct to the London depot at Grosvenor Road or elsewhere along the river as required. Each of the floating plants, seen in *Fig.* 1, is equipped with a Priestman grab of 1½ cu. yds. capacity, which excavates the material from the bottom of the lake and delivers it to the hopper at the top of the erection on the pontoon, whence it falls on to a screen made up of sections each with a different mesh. The screen is inclined in the usual way, and the different meshes arranged so that the sand falls through the top section and larger sizes through lower sections. Anything larger than ¾ in. passes over the screens and is automatically conveyed to a crusher, after passing through which it is taken by an endless chain of buckets to the top of the screen and is again graded. Not only is the ballast practically clean when it is brought up from the bottom of the lake, but throughout the whole

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Loading Depot at Chiswick.

Excavator at Work at Chiswick.

A Modern Aggregate Plant.

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of its passage over the screens it is again copiously watered to prevent it clogging, and this washing on a shaking screen ensures that no particle of foreign matter remains. After passing through the screens, each size of material passes down a separate chute to separate bins at the bottom of the pontoon, and from here it is lifted on continuous bucket elevators to the chutes which deliver it direct into the barges. Each of these floating plants can load a 160-ton barge in from 4½ to 5 hours, and the barges either pass through the lock to the river or are taken to a depot at the side of the lake where they are unloaded and the aggregate taken by rail to a depot on the main road (near Ham Common) about a mile and half away. From this depot deliveries are made by lorries to any point within a 10-mile radius, and it was constructed chiefly for the large supplies required in south-west London. The firm has installed its own electric power plant, with a 200 B.H.P. cold-starting crude oil engine supplied by Messrs. Ruston and Hornsby, Ltd., at the works, and in addition to large stock piles has duplicated the machinery to avoid delay in case of breakdown. An interesting feature of the riverside works is what must be one of the earliest concrete barges made: it was built sixteen years ago, and is still giving satisfactory service.

At Chiswick the Company has been fortunate in securing a large deposit of first-class sand and ballast within a convenient distance of central London, and from here all deliveries are made by road with a fleet of 5-ton Leyland lorries. This site is also on the river-side, and as it was only acquired last year it was possible to lay it out to best advantage to meet modern requirements. The area being worked covers about 60 acres, and is bounded on the one side by the river and on the other by the new 60-ft. wide Gt. Chertsey Road. The depot lies a few yards back from this wide thoroughfare and is reached by a semi-circular approach road, so that lorries enter at one end, load at the centre, and pass out at the other end in a continuous stream without having to turn or back out; each lorry is loaded with 5 yds. of material in four minutes. The deposit consists of a top layer of 5 ft. of sand, and small shingle which is used for tar-spraying roads, beneath which ballast is found to a depth of 16 ft. Excavating operations have been commenced at the water's edge, where a Ruston drag-line excavator, with a bucket of 1½-yds. capacity, mounted on a caterpillar track, excavates the ballast from the river bed at the rate of 60 cu. yds. per hour. From the bucket the material is dumped into jubilee wagons, which are hauled by a petrol engine to the grading plant at the depot. Here it is dumped into pits from whence it is picked up by grabs and placed in the hopper at the top of the grading plant or direct into lorries if it is required ungraded. This plant is on somewhat similar lines to those at Ham. The graded material is delivered through chutes to separate bins, and from these it is loaded either direct into lorries on the approach road or to the stock piles.

At both these works a speciality is made of sand and shingle for reinforced concrete construction, and it is for these materials that the firm is well-known. Ham River materials have been used for some of the most important buildings in the London area, and are playing an important part in the rebuilding now going on. They are also generally used for testing purposes, and are in many quarters looked upon as standard materials.

In addition to supplying aggregates, the Company has excellent facilities for chuting rubbish to fill the excavations at both its works. There is a specially convenient arrangement at Chiswick, where instead of the unmade roads and soft ground so often met with the vehicles dump their contents on to a concrete platform from which the rubbish is picked up by a grab and placed where required in the excavations.

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New Bathing Pool at Tynemouth.

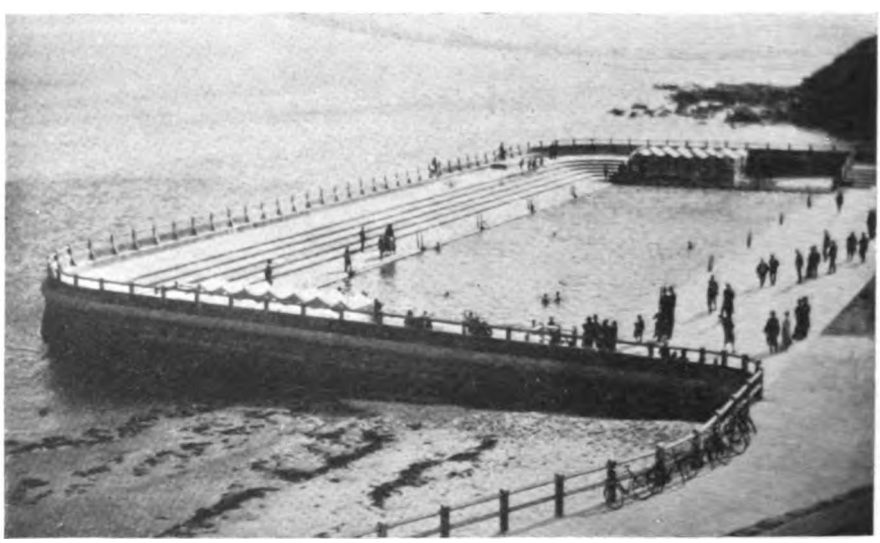
We illustrate herewith a new bathing pool which has recently been completed and opened on the sea front at Tynemouth. The area of water is 180 ft. by 60 ft. The depth ranges from 6 ft. 6 in. to 3 ft. 6 in., the deck around the pond being 6 in. above water level.

The pond is filled and emptied through sluices. For filling there are two sluices at high level so that the sea water is taken in at high water, while the sluice for emptying is at low level for use during low tides. All three sluices open into one chamber, both the incoming and outgoing water passing from the chamber to the pond through a 4-ft. conduit covered with a reinforced concrete grid at its entrance to the pond.

At the seaward side and at each end the pond is protected with a concrete wall faced with pre-cast concrete blocks backed with mass concrete and stiffened with counterforts. The counterforts, and also the walls, are reinforced with old railway plates. The blocks were cast with dove-tail joints in order to lock them together, and also with depressions in the top surfaces in which are laid wire ropes to lace them together. The coping was cast *in situ*. The top of the coping is 4 ft. above H.W.O.S.T., and is surmounted with a parapet railing the lower 2 ft. of which is also closed, so that the top of the enclosure is 6 ft. above H.W.O.S.T., which is sufficient to keep out the sea during the bathing season. To keep out the sea during storms it would have been necessary to build the enclosing wall an additional 10 ft. or more in height, and this was ruled out on the ground of cost. The terraced stands at the sides of the pond provide accommodation for 2,000 spectators.

Difficulty was experienced owing to the presence of a number of small springs on the foreshore, and to prevent these from finding their way below the floor of the pond, with the consequent danger that they might lift it, they have been confined within a restricted area by means of a concrete wall carried down to the shale; within this area the water now rises to a level above high-water mark, and it is drained off through an overflow pipe. The floor of the pond is in mass concrete.

The work has been carried out under the direction of Mr. John F. Smillie, the Borough Surveyor.



[*Mr. J. F. Smillie, Borough Surveyor.*

New Bathing Pool at Tynemouth.

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Design of Formwork for Reinforced Concrete Construction.

By A. E. Wynn, B.Sc., A.M.Am.Soc.C.E.

VIII.—WALL FORMS.

THE same general methods of construction will apply to any kind of wall and whatever the height and location, but it is convenient to consider separately basement and partition walls, low spandrel walls, very high walls or piers, retaining walls, and curved walls, as each kind will have a few distinctive features.

The first thoughts will be whether to build the forms in place, assembling and erection being done in one operation, or in panels where assembly is done at a central point and only erection is done in place; whether to build sufficient forms for the whole wall, or to use the forms a number of times over, and in the latter case whether to raise the forms vertically or to move them horizontally. These questions have to be decided for each individual job, but a few general statements can be made.

It is always well to build sufficient forms for one full day's pour, say, from 40 cu. yds. up, depending upon the size of the mixer. If a wall can be poured in 7 hours there will be at least 1 hour each day left, most of which time will be wasted as the last hour cannot be used efficiently.

On a large job it will probably be necessary to build sufficient forms for two or three days' pour so that the carpenters can always keep ahead of the concreting gang; in this case the forms should be made up in panels for quick stripping and erection.

Small walls that can be poured in one or two days are generally cheaper formed in place, as panel forms will only save money when they can be used several times.

Forms for spandrel walls up to sill line can be used several times, and so they should be made up in panels.

Basement walls and any walls up to about 12 ft. high are generally formed the full height. Above this height they become unwieldy and more difficult to brace, and so are sometimes formed halfway up and the forms raised vertically. It is easier, however, to make a good vertical joint than a good horizontal joint; and cheaper in labour to move forms horizontally than vertically whenever possible, since they are easier to handle. Also, structurally a vertical joint is better than a horizontal one. Very high walls, gravity retaining walls, piers and dams are often built with panel forms 6 ft. 10 in. high, moved vertically. This is not always permitted, however, for engineering reasons, and it is often necessary to form the wall the full height.

Very high wall forms can be moved horizontally, but special handling equipment is then necessary, and they are generally of steel, not timber. "Moving," or "creeping," forms will be described in another chapter. For all walls built in place, and also for panels if not used more than two or three times over, 1 in. sheathing is thick enough. With several usages

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it will pay to use 1¼-in. or 1½-in. sheathing. If particularly good lines are required it is better to use the thicker sheathing.

Basement and Partition Walls.—These will cover all ordinary walls in a building, whether in the foundations or as partitions. They will be from 8 ft. to 12 ft. high, and 2-in. by 4-in. studs are generally used at from 14-in. to 16-in. on centre with 1-in. sheathing.

Note from the design tables that the stud spacing depends upon the allowable span of the sheathing. For a wall 10 ft. high a 3-in. by 4-in. stud can only be spaced 1-in. farther apart than a 2-in. by 4-in. while the same number of wales or ties will be required, and so since a 3-in. by 4-in. will cost considerably more than a 2-in. by 4-in. there would ordinarily be no economy in using the heavier stud. However, if there are to be no exterior braces the heavier stud would be better as it would give a stiffer form.

Note also that the allowable span of the sheathing is based on a deflection of ⅛-in. When the deflection does not matter, as when a wall is covered up, the studs may be spaced farther apart, but the ties should be closer.

Studs need not be cut off at the top of the wall but can be used in random lengths to avoid waste. In all form building an inexperienced man may spend a lot of money cutting up his material to exact lengths resulting in neat but wasteful forms.

If the form is to be built in place, which is the cheapest method for small walls, it should be built as lightly as possible, consistent with strength. The number of nails used should be kept down to a minimum, just sufficient to keep the different members of the forms in place. Nails are seldom used for strength, and they increase the cost of stripping and lessen the salvage value of the timber.

The studs are first set to line all the way through with a board nailed on top and bottom to hold them together, and temporarily braced with 1-in. by 6-in. braces. Lines should always be set an inch or two away from the actual face of the wall so that the forms will not interfere with them. The lowest board should be nailed on level, otherwise the sheathing will tend to run up or down hill, and if the foundation is stepped or uneven the special widths of boards are put on last, not first. The sheathing is nailed on lightly, breaking joints preferably at the studs and the tongue tapped well into the groove so as to get a tight form.

If the wall is narrow and there is much reinforcing steel to place, one side should be completed first, plumbed and braced, and the other side set from this by means of 1-in. by 2-in. spacers cut to the exact width of the wall. The spacers should be about 3 ft. apart vertically and at alternate rows of studs, and are knocked out as the concrete reaches them. If the wall is wide enough for a man to work inside to place the steel both sides may be brought up together and less temporary bracing will be required since the studs can be tied across at the top with short pieces of board.

Only one side of a wall should be accurately set to line, plumbed, and braced, since the other side will be set automatically by the spacers.

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The side to set first depends usually on which is the most important face of the wall, generally the outside if exposed to view and the inside if below ground, also ease of bracing enters into consideration. A good deal of time can be wasted in trying to set and plumb each side independently.

When the form is completed the temporary braces should be replaced by 2-in. by 4-in. or 3-in. by 4-in. braces, as in *Fig.* 24, about 10 ft. apart, the distance depending mainly on how stiff the forms are built. If the wall is low, 1-in. by 6-in. braces will be sufficient, as in *Fig.* 23. Low and unimportant walls can be built without wales, but about 6 ft. high there is always danger of the form twisting and getting out of line from the impact of the concrete if wales are not used. Also wales will transfer the pressure from one stud to another if a tie is loose or breaks. Wales should only be nailed on sufficiently to hold them in place.

When wales are used it is possible with care to do without exterior braces, even with very high walls, but a few are always advisable. The braces should be wedged against the wales, and are very useful if the form begins to get out of line since the wedges can be loosened or tightened to bring the form to line again.

The ties can be inserted before plumbing the forms but should not be tightened up until afterwards; if done beforehand some will become slack and others too tight and they will have to be gone over again.

Ties.—Whether to use wire ties or bolts is always an open question. For very accurate and important work bolts are undoubtedly better, but for ordinary walls in buildings it is almost universal practice to use wires. It is more a question of practical knowledge than theory whether wires are strong enough and how many should be used. It depends mainly on the temperature and the speed of pouring, since in warm weather and with slow pouring the concrete will set fast enough to support itself to some extent and so relieve the pressure on the ties.

If a wall 12 ft. high, for instance, is poured in 8 hours, at least 4 ft. to 6 ft. of it will be set sufficiently by the time pouring is finished to support itself and the wet concrete above to a large extent so that the pressure in the lower ties will be very much reduced from what it would be if the wall were poured in 2 hours. It has been seen in the previous chapter, Design 13, that for a wall 8 ft. high two double strands of No. 9 wire, or 4 wires, are required on the lowest wale, assuming maximum pressure.

In general, for walls up to 12 ft. high, with 1-in. sheathing and every stud tied, two double strands of No. 9 wire should be used on wales in the lower half of the wall and one double strand on wales in the upper half of the wall. With thicker sheathing, and when only alternate studs are tied, bolts should be used and their size calculated. When wales are used the wires can pass around stud and wale or around the wale only. The holes are bored from one side of the wall only, using an extension bit to bore through the far side.

The wires should be twisted up tight inside the wall, usually with

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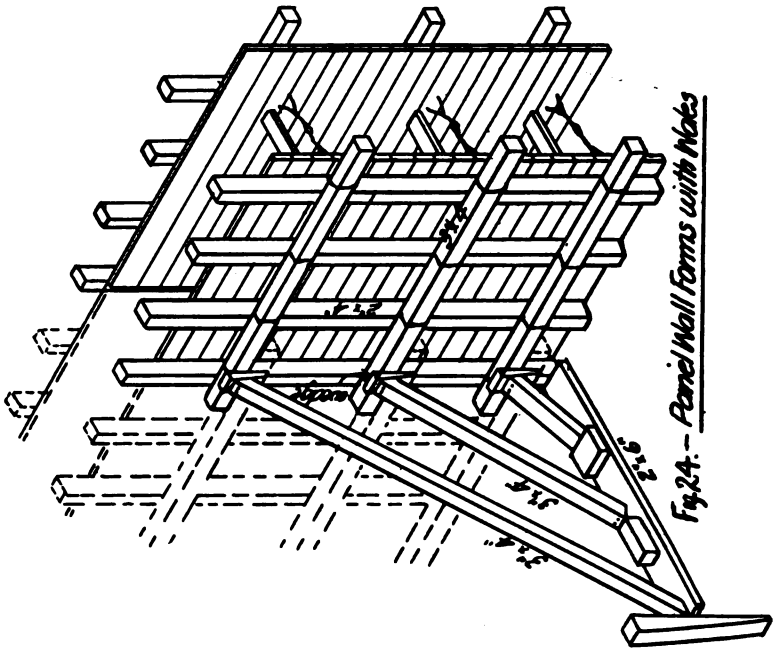

Fig. 24.—Panel Wall Forms with Wales

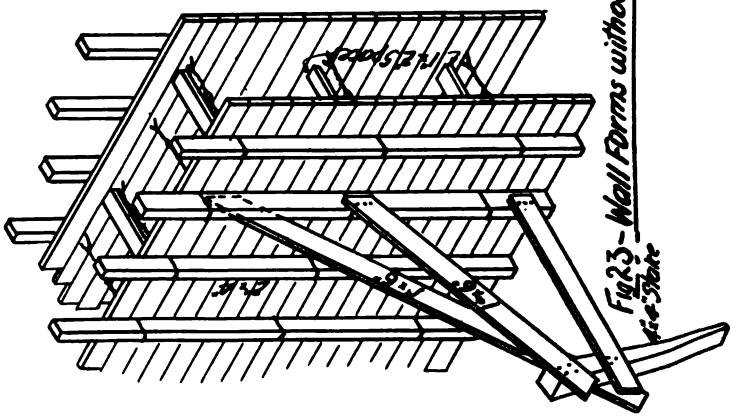

Fig. 23.—Wall Forms without Wales

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the claw end of a hammer, but not so tight that they are on the verge of breaking, and there should be a spacer just above each wire to draw the forms up against. The spacer should be above the wires, as they are commonly used for walking on and placing them above prevents walking on the wires. The wires should be hammered flat against the wales or studs, before tightening up, or wedges driven in after tightening, otherwise the slack will be taken up under pressure and the form will give slightly. Two men should work together on the wires, one each side the wall.

Bolts are not used with single studs and no wales, but if a stud is made up of two pieces the bolt is placed between them, bearing on a short piece of timber. When wales are used the bolt is placed through the wale only, close to the stud, to save boring through the stud.

If a contractor is doing a great deal of concrete work he will generally find plain round rods and patent clamps more economical than bolts, since a smaller size can be used in the first place and they can be ordered long enough so that they can be used many times over in walls or columns. They are more easily drawn than bolts, and if they have to be left in the wall their loss is less. Bolts, to be adaptable, must have long threads, because of the different thicknesses of wall, or else several lengths must be carried, while rods 3 ft. or 4 ft. long can be used almost anywhere. The initial cost of clamps, however, does not warrant their use unless they can be used several times over.

Wires are cut off a little within the surface of the concrete, so that they will be covered when finishing and there will be no danger of rust spots. Bolts and rods are more often drawn than left in, so they should be greased before pouring. They should be withdrawn within two days after pouring, before the concrete has set too hard to make it impossible or expensive. Rod pullers can be bought for this purpose.

There are many devices, mostly patented, for enabling the ends of a tie to be unscrewed leaving the centre of the tie in the wall, and as their main advantage is that a hole is not left right through the wall it may be necessary to use them when a wall has to be absolutely water-tight. A simple device of this kind consists of a rod a little shorter than the width of the wall, threaded each end, and fitted with unions into which short pieces of rod are screwed. These end pieces are easily screwed out and the holes plugged (*Fig.* 29). To withdraw a bolt easily an iron pipe-sleeve a little shorter than the width of the wall and fitted each end with a wooden washer can be slipped over the bolt.

The disadvantage of special devices is that they generally have to be ordered for a certain width of wall and so are not adaptable and also a part will be lost in the wall. A greased rod, and even an ungreased rod, will easily pull out of a green wall if given a twist to start it and can be used again many times.

Panel Construction.—On a large job where wall forms can be used several times over it will be best to build them in panels. This has also the advantage that the panels can be made up in advance, so saving time in erection. Their size will depend on the facilities for handling

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them ; 8 ft. by 10 ft. is a convenient size, weighing about 550 lbs., and can be handled by four to six men.

Wales are generally put on during erection and the holes bored at that time, and the wales should be long enough to overlap the end stud of the adjoining panel. The ends of the sheathing must, of course, be cut off square ; at one end of the panel there will be no stud, and at the other end the sheathing will finish at the centre of the stud, so that the next panel can join on to the same stud (*Fig.* 24). One side of the wall will be set to line first, plumbed, and braced. The spacers will be lightly tacked on to this form and the other side up-ended against the spacers, bolts or wires inserted and drawn up tight against the spacers.

As these forms will have to withstand much more handling than when built in place, 1¼-in. sheathing and 3-in. or 4-in. by 4-in. studs are better than the lighter sizes. The studs in panel construction should preferably be of the same length as the height of the panel. Wall panels can sometimes be used again in the floor construction, and can be built in the first place with that in mind. The panels should be oiled before being erected.

Double Studs and Wales.—For light work some contractors use double studs consisting of two pieces of 1-in. or 1¼-in. by 4-in. instead of a solid 2-in. by 4-in. The two pieces are separated by 1-in. thick blocks about 3 ft. apart, forming a slot through which the bolt is passed (*Fig.* 28). A short piece of 2-in. by 4-in. must be slipped over each end of the bolt to give bearing for the washers. Wires cannot be used with double studs. The forms are made in panels, the sheathing being nailed together with 1-in. by 4-in. battens at the same spacing as the studs, which in this case are not nailed to the sheathing. The panels are erected, plumbed, and braced, the holes bored, and the built-up studs placed in position and bolted up tight against the spacers, spacing the studs between the battens.

Although there may be a small saving in timber cost, the labour will cost much more than with solid studs as there are so many more pieces to handle. There will also be less salvage with the lighter timber, so that using double studs is doubtful economy. There may be some economy, however, in using double wales, especially when a heavy size is required, such as a 4-in. by 6-in., when two 2-in. by 6-in. may be much cheaper in material and the labour cost will be little more. The bolts, as before, are inserted between the two pieces, and wood bearing blocks need not be used if the waling pieces are 2 in. thick or over (*Fig.* 29).

Column Offsets.—Basement walls often have offsets where the columns are carried, or a thin partition wall may be poured with the column giving a projection on each side. The first can be built as in *Fig.* 30. The face of the projection has the sheathing extended to carry the stop battens to which the sides are nailed.

In the second case an ordinary column form is built with the end panels cut to leave an opening equal to the thickness of the wall plus twice the thickness of wall sheathing. The wall sheathing should stop about ¼-in. from the inside of the column sheathing, and should have bevelled edges to allow for give in the column forms (*Fig.* 31).

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Bulkheads and Keys.—When pouring a wall in vertical sections a bulkhead has to be put in at the end of each section and a key left for the new concrete. This is constructed as in *Fig.* 27. If the wall is wider than the allowable span of the sheathing a centre stud must be used and tied diagonally to the side studs or braced from the outside. For a heavier wall a larger key should be made of 1-in. boards nailed to 1-in. blocks bevelled two sides (*Fig.* 27).

Window Openings.—When there are basement windows in a wall a four-sided box is built the size of the window opening and nailed into the form (*Fig.* 26). For small walls the box usually consists of 2-in. plank, and for larger walls of 1-in. sheathing, on 2-in. by 4-in. frames. Around the outside of the box must be lightly nailed a 2-in. by 4-in. nailing strip to which to nail the wood window frame, or if steel sashes are used to form a groove into which to grout the sash. With steel sashes the window sills are usually poured after the sashes are set, so

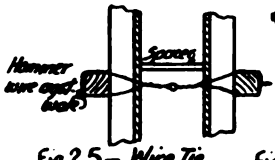

Fig 25 – Wire Tie

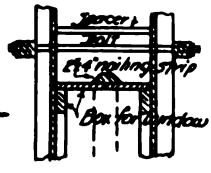

Fig 26 Window Opening

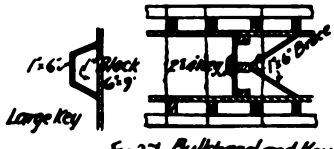

Fig. 27 Bulkhead and Key

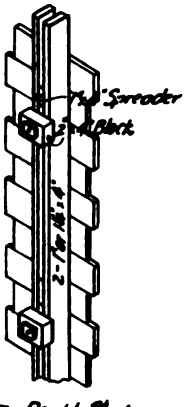

Fig. 28 – Double Stud

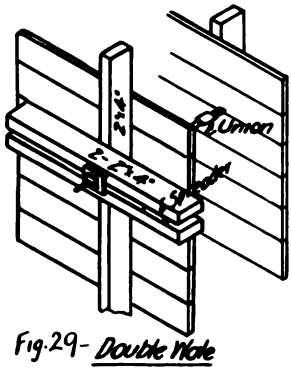

Fig. 29 – Double Wale

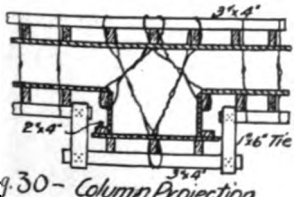

Fig. 30 – Column Projection

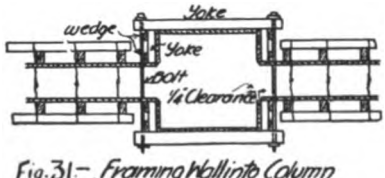

Fig. 31 – Framing Wall into Column

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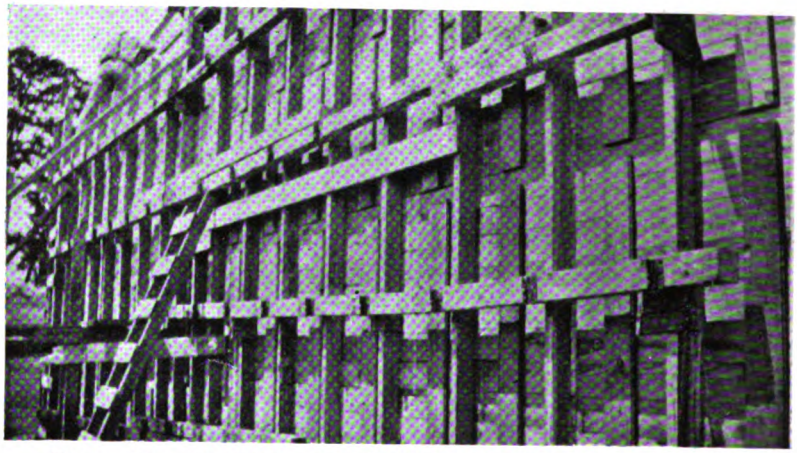

FIG. 32.—PANEL WALL FORMS USING INTERMEDIATE BATTENS AND PATENT WIRE TIES.

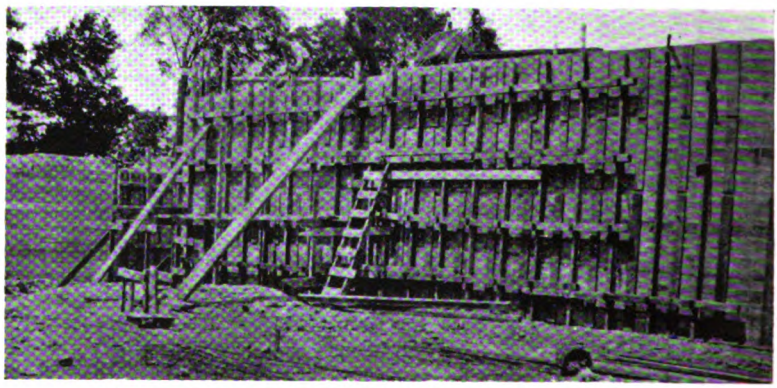

FIG. 33.—WALL PANELS, 18 FT. HIGH; THREE PANELS VERTICALLY

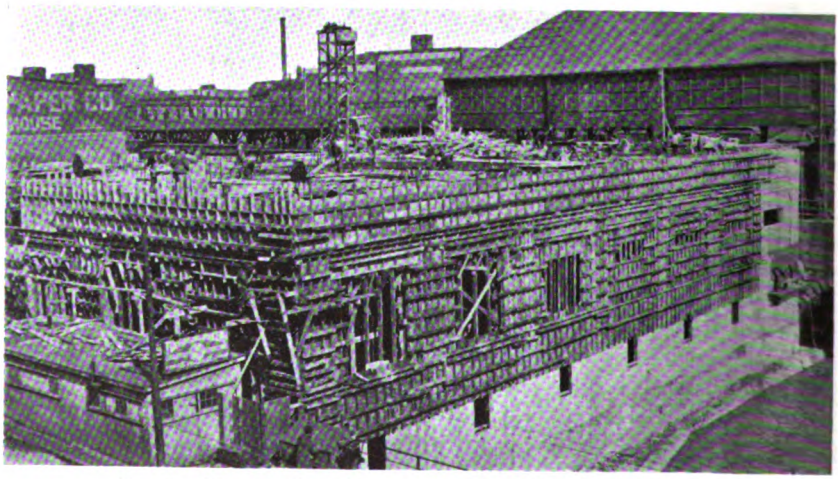

FIG. 34.—DIFFICULT WALL FORMS FOR AN EXPOSED CONCRETE EXTERIOR.

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that the size of the box must include the depth of the sill. The nailing strip must be nailed on lightly so that it will remain in the concrete after stripping the box. To make stripping easier the box is often sawn through the middle of each side and the halves joined with battens, which are taken off before stripping.

Corner Bracing.—The weakest part of a wall form is usually at the corners; there being no studs directly opposite the corner one, tying or bracing this is often forgotten. Wales should overlap at the corners and be spiked together, and the outside corner studs should be tied diagonally across to the inside corner stud either separately or with wires around both studs.

Single Wall Forms.—A wall poured against an existing wall or against the bank of an excavation needs only one form, and this must be braced externally. If there is anything close to brace to horizontal braces are best, but usually they have to be taken diagonally down to the ground where they are held by stakes. In order to be able to adjust and plumb the form, wedges are used between the brace and wales, driving a nail through both to prevent slipping (*Fig.* 24). If there are any signs of the wall going out of plumb the nails are withdrawn, the wedge driven up, and the nails replaced.

If no wales are used, 1-in. by 6-in. for low walls and 2-in. by 4-in. for high walls are used as braces at each stud and are nailed to stakes (*Fig.* 23). With wales the braces are generally 3 in. or 4 in. by 4 in. and they can take bearing on a 2-in. plank or 4-in. by 4-in. laid on the ground with a stake at the end (*Fig.* 24). If it is necessary to brace to a finished floor the 4-in. by 4-in. can be held with stud bolts as for columns.

Low Spandrel Walls.—These walls span from column to column

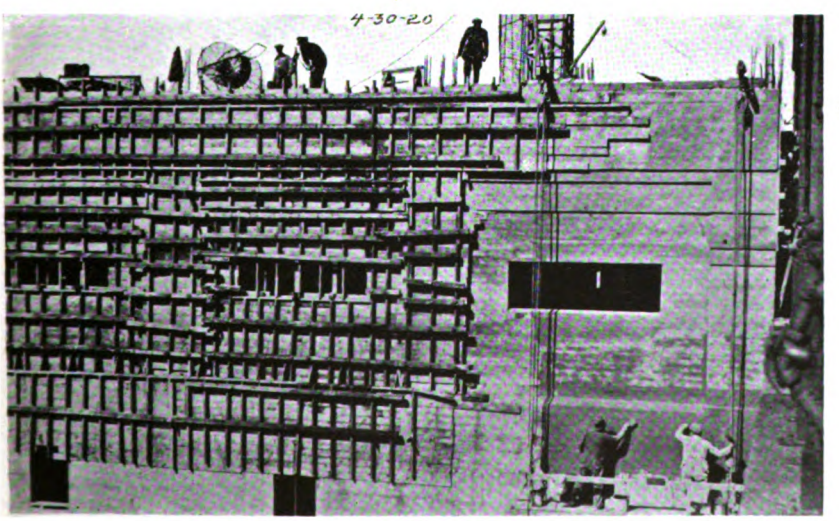

Fig. 35.—Wall Forms for Ornamental Exposed Concrete Exterior.

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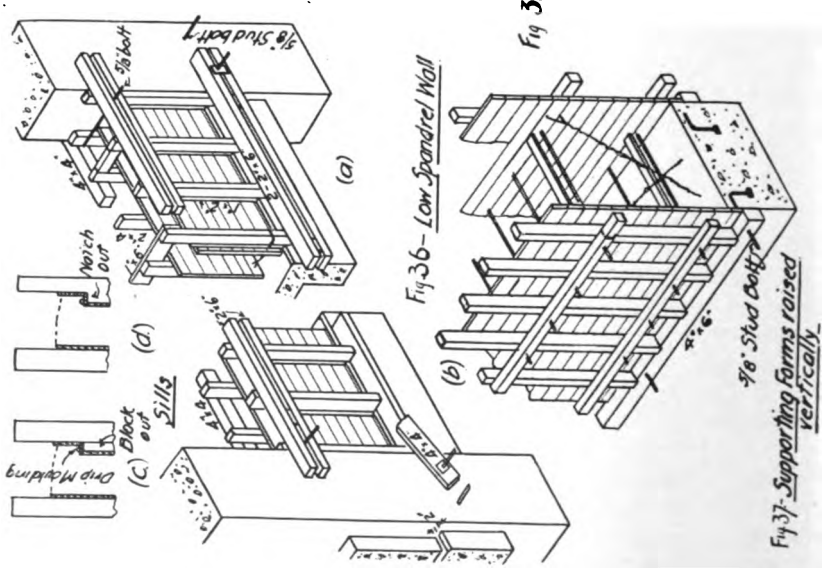

Fig 36 - Low Spandrel Wall

Fig 37 - Supporting forms raised vertically

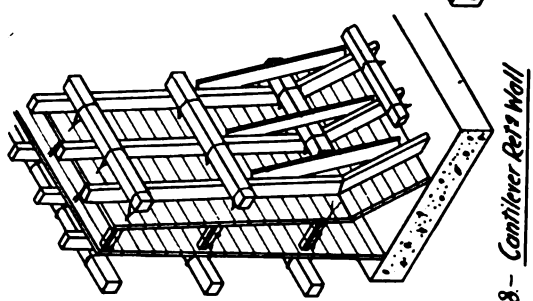

Fig 38 - Cantilever Retg Wall

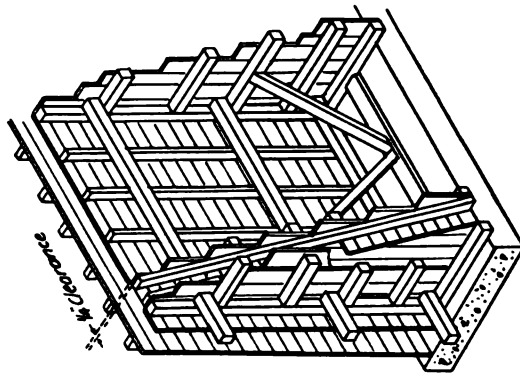

Fig 39 - Counterfort Retg Wall

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and are only sill high. As there are generally several walls of the same height and length the forms are made up in panels of 1-in., or preferably 1¼-in., sheathing on 2-in. by 4-in. studs. They are built after the columns are stripped, so that the main problem is the method of bracing them.

To hold the bottom of the outside form the stud bolts left in the column 6 in. below the floor line can be utilised. Two 2-in. by 6-in. separated by 1-in. blocks can be slipped over these bolts and bolted tight to the column. These will hold the studs, which must be about 8 in. longer than the sheathing. If the wall is flush with the lintel the sheathing should extend an inch or so below the floor. If the wall is set back from the lintel it may be necessary to wedge between the studs and wale (*Fig.* 36*a*).

Another method is to nail a 2-in. by 6-in. on to the bottom of the studs and to hold it by short pieces of 4-in. by 4-in. over the bolts (*Fig.* 36*b*). At the top a similar wale is bolted through close to the columns to a 4-in. by 4-in. on the other side of the columns, long enough to hold the first two studs. The inside form is held at the bottom by wires or bolts in the ordinary way, and the top by nailing short pieces of 1-in. by 6-in. across the studs or by wires.

Sills are usually poured after the walls are stripped and the sash set, but if the sash is set before the walls are poured the sills and walls can be poured together. To form the sill projection the studs can be blocked out from the sheathing 1 in. or 2 in. as required, or notches can be cut in the studs (*Fig.* 36).

Very High Walls and Piers.—If a high wall is to be formed to the top the methods will be the same as already described. Timber sizes need not necessarily be heavier if care is taken, but since it is difficult to brace high walls from the outside heavier timber will give a stiffer form, so 1¼-in. sheathing and 4-in. by 4-in. studs are preferable. Wales should always be used, and bolts or rods instead of wire, except perhaps near the top.

High walls and piers are often built by raising the forms vertically in panels if horizontal joints are not objectionable. The panels are made the same as when they are moved horizontally, but they should be about 6 in. higher than the height of pour in order to lap over the concrete already poured. One lift is all that can be managed with hand-power, using a hand derrick. With derricks, cranes, cableways, etc., very high lifts can be made. A panel 8 ft. by 10 ft. is convenient to handle, but with power handling larger panels may be used.

To support the bottom of the panel after it is raised, stud bolts or rods should be left in the wall on each side at each pour, 6 ft. to 8 ft. apart and about 9 in. down from the top of the pour (*Fig.* 37). To these is bolted a 4-in. or 6-in. by 6-in., which acts as a support for the panels. To hold them temporarily while being plumbed, wires can be embedded in the concrete, twisted around the reinforcing steel if possible, and sufficiently long to reach to about the centre of the studs. The first wale and bolts should be just above the concrete, so that the forms can be drawn in tight against the concrete to avoid leaving a ridge at

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the joint. Using spacers in the wall there will be little difficulty in plumbing the forms. High walls built with moving forms will be mentioned later.

Retaining Walls.—Plain concrete walls will be built by one of the methods described, but reinforced concrete retaining walls may have a few special features.

With cantilever walls there is often a fillet or bracket at the bottom of the back of the wall (*Fig.* 38). The form for this is built as a separate panel with the same spacing of studs as for the wall itself. As there will be upward pressure on this panel it must be well anchored down, generally by wiring to the reinforcing steel. The wall panel is set on top of the sloping panel and the studs of each nailed together. There should be a spacer at the bottom of the wall panel. A diagonal 1-in. by 6-in. brace is nailed from the sloping to the vertical studs to help support and stiffen the wall form. The first pair of wales will be at the junction of the sloping and vertical studs, but if this is more than about 2 ft. high the bottom of the studs must be held by wales bolted through the wall or braced from the outside. As the back of the wall will usually be on a batter, the spacers will vary in length. The top and bottom spacers must be cut to the exact width of the wall and are set first spacing the top and bottom of the forms. The intermediate spacers are cut approximately to suit the width and are wedged into the forms to hold the panels to line.

Counterfort walls present no particular difficulty. The sloping back of the counterfort is built similarly to a bulkhead. For easy stripping it should be built as a panel with 1-in. by 4-in. battens down the centre and only lightly nailed to the stop battens on the sides, as the pressure of the concrete will keep it in place. The sheathing of the sides of the counterforts should not be cut to the slope, but should be of random lengths to save material. Since counterforts are usually about 8 ft. apart they are easily braced against each other with diagonal braces butting against a stop block nailed to a 2-in. plank or a 4-in. by 4-in. which holds the lowest wale (*Fig.* 39). When the counterforts are short it is usually better to run the sheathing vertically. The braces should not be horizontal between counterforts unless the concrete is brought up in each one at the same rate. As the counterforts will be stripped first the wall sheathing should overlap the counterfort sheathing (*Fig.* 39).

Curved Walls.—These will have vertical sheathing and horizontal yokes like a circular column (*Fig.* 40). The curve should be laid out on the ground by driving in a stake at the centre and marking out the radii with a lath nailed to the stake and with a hole at the other end for a pencil. Along the lath is measured the outside radius plus the sheathing thickness and the inside radius less the sheathing thickness. Along the circumference are loosely placed 2-in. planks the width depending on the radius, lapping 6 in. to 12 in. The lath is swung around over the planks, which are adjusted until the laps are about equal and the depth of cut at the ends the same, when they are nailed lightly together and the curve marked out on them. They are then taken

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apart and the curves cut out. When put together again with the same amount of lap they will give the correct radius.

When possible it will save timber to cut both inside and outside yokes from the same plank and to do this it will pay to use wider and shorter plank. The inside yokes should be marked out first and then the planks taken apart and moved farther out until the greater outside curve coincides with the inner curve at the centre of the yokes. The outside yokes must be longer than the inside yokes because of the greater curvature if each are lapped the same amount, the ratio of the lengths being approximately the same as the ratio of the outside and inside radii. The inside yokes should be cut back at each end to avoid feather edges. Sometimes only one plank is marked out in this way and used as a template for cutting the others, but if there is a slight error in the template this will be repeated in all the yokes and they may not fit together so well.

Instead of using 2-in. planks two thicknesses of 1-in. boards can be used nailed together, with the splice in one board at the centre of the other board as shown in the dotted lines in *Fig* 40. This is more suitable for inside than outside yokes. The least depth of outside yoke at the centre should be not less than 4-in., nor the end depth of inside yokes less than 4 in. at the centre of the lap.

When the yokes are cut out they are nailed together at the laps, the radius being checked again before driving home the nails. The sheathing is nailed on either in place or in panel form. The sharper the curve the narrower must be the sheathing.

The spacing of the yokes depends on the allowable span of the sheathing (Table 2). When erected the forms are wired across at the laps of the yokes and at intermediate points so that the wires are not over 30 in. apart, depending on the height of the wall. If the wall is high, vertical wales should be used at the ties to stiffen the form.

With very flat curves it is often possible to spring the boards horizontally to the curvature. In this case the lowest board is staked to line and the studs then nailed on. No wales can be used without wedging out from the studs, so there should be some outside bracing.

If a wall is a complete circle, as in tanks and silos, and often in small foundations, it is possible to dispense with the ties, but in this case the timber of the outside yokes is in tension and the nails or bolts at the lap of the yokes in shear. Since it takes about ten 20*d*. spikes to equal one $\frac{5}{8}$-in. bolt in shear, bolts should be used to fasten the yokes together at least two to a joint. If the wall is very low and of small diameter, five 20*d*. spikes can be used at each joint instead of the bolts. Concreting must be brought up equally around the wall, otherwise the forms will be thrown out of shape.

Batter-Curved Walls.—Curved wing walls for bridges, bridge piers, foundations for turbines, etc., are often battered. The yokes at different heights will have different radii, which must be calculated. The radius at the bottom will be greater than that at the top, so that the width of each board should really taper from bottom to top. This, however,

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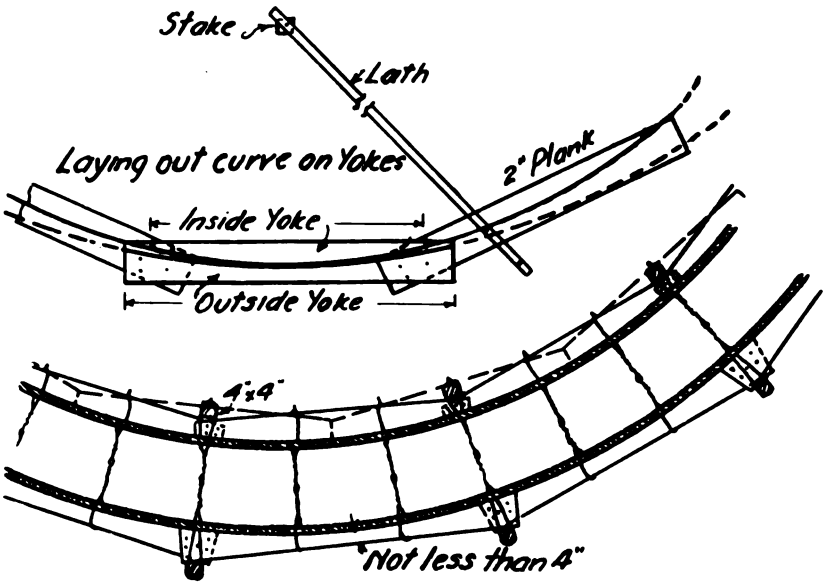

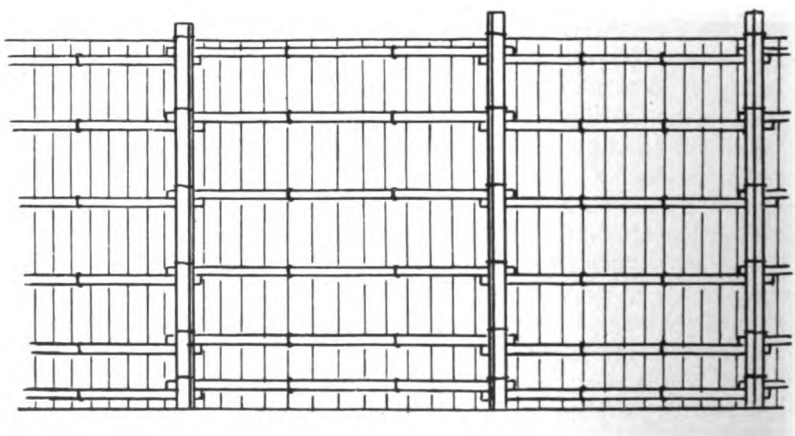

Fig. 40- Curved Walls

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is seldom done. General practice is to nail on three or four boards of the regular width, leave a space at the top of about 2 in., then nail on some more boards, leave a space, and so on. Each space is then filled with a special board cut to fit. It is best to use square-edged boards with this method so that the wedge pieces can be slipped in without having to cut off the tongues (*Fig.* 42).

Stripping.—After the exterior braces are removed the wires are cut or the clamps or nuts removed. There should be a box handy in which to drop the parts, otherwise a large number will be lost. If the forms have been built in place, the wales are then knocked off and the studs pried away individually from the sheathing; the advantage of using as few nails as possible will be seen here. As the studs have to be taken apart from the sheathing, it is easier to do it when they are standing in position rather than try to take sheathing and studs down together and then remove the studs; there is also the advantage of being able to pile the studs together without sorting them out from the sheathing. The sheathing boards are then pried from the wall, and lastly the wires cut off or the bolts or rods pulled. Sometimes the bolts are pulled before stripping, but generally the timber is required first for use elsewhere and so the bolts are left until last.

In panel construction the panel is removed as a whole, prying out from the top of the wall. In this case the bolts are generally removed first, as the panel can be stripped much more easily, but if not the wales are knocked off before removing the panels.

In the summer forms can be removed in from 12 to 24 hours, but in cold weather they should be left 3 or 4 days, or until the concrete

Fig. 41.—Circular Wall Form: Showing Method of Bracing.

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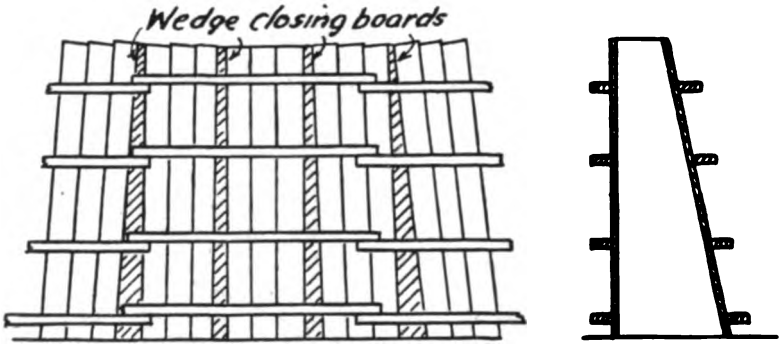

Fig. 42— Battered Curved Walls

cannot be indented with the thumb nail. The timber should be cleaned of concrete as soon as possible after stripping, as hardened concrete is difficult to remove. Different sizes should always be piled separately, as much time can be lost in searching for studs under a pile of sheathing boards. Regular carpenters' stripping bars should be used for stripping, and when it is necessary to pry from concrete a short board should be slipped between the concrete and bar and care taken not to break off edges.

ESTIMATING COST.

The amount of timber required per square foot of contact area will increase with the height of the wall if the whole height is poured at one time, and will depend on whether the timber sizes are the minimum required for strength or whether they are made heavier to allow for wear and tear.

It will vary from a minimum of 0·175 cu. ft. to a maximum of 0·35 cu. ft. per sq. ft. of contact area, measuring both sides of the wall. For ordinary building walls up to 10 or 12 ft. high an average of 0·225 cu. ft. per sq. ft. can be used. For high retaining walls of 20 ft. and over 0·30 cu. ft. is an average value, and proportionally for intermediate heights.

If the forms are built in panels and used several times these units will be divided by the number of times used, based on the whole area of both sides the wall. But for each time the panel is used 10 per cent. should be added to the amount allowed to make up the panels to cover possible repairs. That is, if the first erection takes 0·25 cu. ft. per sq. ft. and the panel is to be used four times the amount of timber to estimate is $\frac{0{\cdot}25 + 30 \text{ per cent.}}{4} = 0{\cdot}08125$ cu. ft. per sq. ft. This limits the number of times the original timber can be used to ten. However, a panel is not usually used more than six times. These units will include waste in cutting, braces, spacers, etc.

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The labour cost will also vary with the height, but is fairly constant for ordinary walls up to 12 ft. high. Above this height there will be an additional cost for raising the timbers, splicing the studs, and for the inconvenience of working above ground. For ordinary heights the labour cost per cubic foot of timber will be fairly constant, that is the cost per sq. ft. will increase with the amount of timber used per sq. ft., because there will be more weight to handle.

The thickness of the wall does not have much effect on the cost, though very narrow walls, such as 6 in. or 8 in., or very wide walls 3 ft. and over, will generally cost slightly more than the usual widths of 12 in. to 30 in., because in the narrow walls the carpenters cannot work inside the wall and in the very wide walls the ties will take a little longer to place.

Panel forms will cost more than forms built in place if only used once, but if used more than once will cost less.

For long retaining walls, where the forms can be used many times over, the cost of renting steel forms should be investigated.

Allowance for the cost of cleaning and oiling must be made each time a panel is used. A labourer should clean and oil 100 ft. super per hour.

When the forms are done with an additional cost for pulling all the nails must be added before the timber can be salvaged for use on another job. This is often overlooked, though it may amount to a considerable sum on a large job. A labourer should clean timber of nails at the rate of about 10 cu. ft. per hour and the cost should be figured on the amount of timber it is expected to salvage for use on the same job or on another job.

The cost of nails, wire, and bolts is usually allowed for by a lump sum, rather than reckoning the cost for each item of formwork. The number of wire ties required will be approximately the total contact area divided by 1½. If bolts or rods are used this number should be divided by the number of times they can be used plus an allowance for waste and loss. About 1 lb. of nails per 10 ft. super of forms should be allowed, as the waste is large.

For walls up to 12 ft. high built in place it will require about 6 hours carpenters' time and 3 hours labourers' time to erect and brace 100 ft. super of forms, and 1 hour carpenters' time plus 2 hours labourers' time to strip them. To build 100 ft. super of panel forms will require about 3 hours carpenters' time plus 1½ hours labourers' time to make the panels, 3½ hours' carpenters' time plus 3½ hours labourers' time to erect them, and ½ hour carpenters' time plus 2 hours labourers' time to strip.

In either case the labourers' time cleaning must be reckoned each time the timber is used.

Cost of 100 *ft. super of wall forms built in place—*

7 hrs. carpenter @ ..... =
6 ,, labourer @ ..... =
Timber, say, 0·225 × 100 = 22·5 cu. ft. @ =

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Cost of 100 ft. super of wall forms built in panels and used 4 times—

1st erection : 7 hrs. carpenter @ ..... =

8 „ labourer @ ..... =

Timber, 0·25 + 30 per cent. = $\frac{0{\cdot}325 \times 100}{4}$ = 8·125 cu. ft. @ .... =

For each of following erections :

4 hrs. carpenter @ ..... =

6½ „ labourer @ ..... =

Or average cost for whole job =

4¾ hrs. carpenter @ ..... =

6⅞ „ labourer @ ..... =

8⅛ cu. ft. @ ..... =

(*To be continued.*)

BOOK REVIEW.

Building Construction Plates for Students and Others. By A. Buchanan and W. H. Hudson. Part I, Elementary Course, 5*s.*; Part II, Advanced Course, 5*s.* 6*d.*

London : B. T. Batsford, Ltd.

THESE two volumes of instructional diagrams are primarily intended for use in classes in connection with the theory of constructional arts and crafts, and should prove of great benefit both to the students and to the lecturer. Although they are not provided with explanatory text, the diagrams are in most cases so admirably clear that an observant reader examining them with the help of scale and dividers cannot fail to learn a great deal of the normal practice of English building construction. Opposite each plate, however, a blank sheet is provided for "student's class notes" so that the authors, the lecturer, and the students may combine to produce a reference work of great value. Binding the plates instead of publishing them loose in a wrapper also tends towards their preservation for reference.

The nature of the work does not permit of any great flights of originality, but an alternative method is sometimes given in addition to the old-established practice. On plate 9, which is devoted to the illustration of fireplace openings, chimneys, etc., a reinforced concrete lintel to the fireplace opening is suggested, although the cambered iron chimney bar is also shown. The absurdity of the usual upward camber of the chimney bar, which is diminished when the weight of the chimney comes upon it with consequent increase of span, is contrasted in the diagram with the proper downward drooping curve of the reinforcement in the concrete lintel. On plate 22 also modern methods of forming reinforced concrete hearths are placed side by side with the old-fashioned trimmer arch of brick. The use of reinforcement in connection with ordinary brick walling and partitions is indicated on plates 5 and 23. Adherence to precedent accounts for the illustration of the complicated arrangement of ill-assorted roofs on plate 36, and it is to be hoped that the students will realise that this jumble of roofs of different pitches is purposely designed to exhibit a great many different details of plumbing practice.

The first volume contains 46 plates of diagrams relating to the building crafts, one dealing with mensuration and three of builder's memoranda. The first plate is devoted to lettering and, unfortunately, of the eight alphabets presented not one is of outstanding merit. A good way to learn lettering for practical draughtsman's use is to copy the inscription on the Trajan column from the cast at South Kensington Museum and memorise the proportions of the letters. The second volume contains 64 plates and continues the useful informative style of the first. It includes, together with the purely constructive details of concrete, steelwork, masonry and timber, several diagrams concerning sanitation, hot and cold water supply, ventilation and electric wiring The graphical calculation of stresses in steel trusses and girders is also illustrated. —W. H.

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The Foreman's Guide to Concrete.—IX.

A MANUAL FOR ALL ENGAGED IN CARRYING OUT CONCRETE AND REINFORCED CONCRETE STRUCTURES.

By Albert Lakeman, M.S.A., M.I.Struct.E.

REINFORCED CONCRETE COLUMNS.

COLUMNS are called upon to support the ends of beams and provide the reaction necessary to counteract the load, as previously described, and in the ordinary way they are subject to direct compression; that is, the stress is not caused by leverage but simply by the direct thrust downwards of a load placed on the top. In many cases the column will be supporting other columns above, in addition to the ends of beams, and the total load has to be provided for in the strength of the column.

It might appear a very simple matter to design a column subject to direct compression, as there is no complication of bending moments, nature of loads, and similar matters, but there is one point which has to be dealt with that changes the whole aspect of the problem, and this point is *flexibility*. The explanation of the subject in the language of the designer would serve no good purpose in a treatise such as this, as the subject of column design is one that is considerably more complicated than that of beam design, because the stiffness of the column affects its value as a load carrying member and the formulæ used in calculations are based on experiments and not on actual reasoning.

As a simple illustration, let us assume that cane will safely carry a load of 200 lbs. per sq. in. when under compression and then consider an ordinary cane walking-stick which has a sectional area of 0·25 sq. in. This, theoretically, should safely support a load of 50 lbs., and yet if quite a small pressure is exerted at the top when the bottom is resting on the ground it will bend under the pressure, while a load of 50 lbs. would cause it to double up entirely. If, however, a piece of the same walking stick 1 in. long were placed on the floor and a weight of 50 lbs. balanced on the top the cane would support the load quite easily. This illustrates the difference between long and short columns; the walking-stick would fail on account of its flexibility and not because the direct compression was excessive. Theoretically there is no reason why the cane should bend if the pressure is applied directly on the centre of the stick in a direct straight line downward, but in practice it will do so, and thus the theoretical reasoning must give way to actual practical experiment before a reliable method of calculation can be found.

Theory must follow practice, and it has been found by experiment that columns can be divided into (*a*) short columns and (*b*) long columns according to the ratio between the height of the column and its diameter. If the column is rectangular or of irregular shape then the width of the least dimension must be taken for the ratio.

As an example, if a reinforced concrete column is 12 ft. high and 12 in. square then the ratio is 144 in. divided by 12 in., which equals 12; whereas a column 12 ft. high and 12 in. by 6 in. in section has a ratio of 144 divided by 6, which equals 24. If the rectangular column were increased to 14 in. by 6 in. the ratio would still be 24, but if the size were changed to 9 in. square the ratio would be altered to $\frac{144}{9} = 16$. The lower the ratio the stiffer the column and the greater the load per sq. in. that can be safely carried, and the full working stress of 600 lbs. per sq. in. on the concrete is only permitted when the ratio for a square column does not exceed 15 and in the case of a round column when the ratio does not exceed 12. These are known as "short columns," and for all ratios over these the columns are "long" ones and the pressure per sq. in. that is allowed is gradually decreased as the ratio becomes greater.

It will be observed that the term "short column" does not necessarily mean it is a low one in actual height, but that the ratio is low; thus a column 30 ft. long will still be classified as a short one if the diameter is made as much as 30 in.

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In addition to the consideration that has to be given to the ratio of length and diameter, the stiffness of a column is affected by the condition of the ends as regards the fixing. If the ends of a column are rigidly held in position in a manner which prevents any movement under the load it will be less flexible and therefore able to offer more resistance to the weight which is tending to cause it to buckle than is the case when the ends are free to move. It might be considered in the first instance that the ends of all columns are fixed, but this is not correct if the fixing merely consists of attachment to a beam or other member which is liable to be distorted under the loading by twisting or sideways movement. In the regulations of the L.C.C. it is stated that a pillar or strut shall be deemed to have fixed ends when the ends are sufficiently secured to other parts of the construction having such rigidity as will maintain the axis at the ends in its original position and direction under all loads less than the crippling load, and this is probably as simple a definition as can be given without actually specifying the relation between the disposition and sizes of the column and the other members of the structure. This is obviously impossible when dealing with regulations to cover the design of structures generally.

When the ends of a column are free to move under the action of the load the tendency is for the column to bend in one simple curve, as explained for beams that are supported at the ends, whereas the fixing of the ends means that the bending is restrained and the tendency to bend is reduced in a similar manner to that given in connection with beams having fixed ends. When dealing with a column that will theoretically have fixed ends the designer makes an allowance for this condition, and the amount of concrete and steel will be less than that provided for one with free ends; this point is an important one because the work must be executed with accuracy and care in order to produce the condition that influenced the designer when calculating the materials necessary, otherwise the necessary stiffness will be missing and the column will be overstressed.

The length of the column is considered in relation to the conditions at the ends, and the ratio given above for short columns will only be applicable where both ends are fixed in position and direction because a partial fixing only will result in the same condition as an increase in the length, and the length taken when working out the ratio is the equivalent length (described in the L.C.C. regulations as the "virtual" length) and not necessarily the actual length. If the work is not properly executed in the case of a column that should have fixed ends the foreman may unconsciously produce a "long" column in place of a "short" one during the manufacture and the whole of the calculations will be upset. A manufacturer of iron pipes would not be allowed to provide spigot and socket joints when flanged joints were called for on account of the conditions given not being in accordance with the requirements of the engineer, and the joints at the ends of columns are equally important; in the one case, however, the change in the conditions will be obvious by simple inspection after manufacture, whereas in the other case inspection during manufacture is necessary because the work cannot be judged when completed. This throws the full responsibility on to the foreman and the clerk-of-works on the site of operations, and because of this responsibility it is necessary that they should understand the full significance of the details which are supplied by the designer.

There is still one other point that must be mentioned as having a bearing on the design of columns, viz. the links or binding which are provided around the main vertical steel reinforcement. In the absence of any instruction the foreman may consider that these links are provided solely for the purpose of keeping the main steel rods in position during the placing of the concrete, but this is not correct as the links or binding are a definite factor in the strength of the column and the stress on the concrete may be safely increased to a certain extent where sufficient binding is provided. The regulations of the L.C.C. state that the volume of the binding shall not be less than 0·5 per cent. of the volume of the hooped core. This amount is necessary to allow the concrete to develop the working compressive strength of 600 lbs. per sq. in., but if more binding is provided the working stress can be increased up to as much as 800 lbs. per sq. in. for an ordinary 1 : 2 : 4 mixture

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according to the percentage of binding to concrete.

Again this point is of considerable importance to the foreman because the designer may provide links at short intervals with the definite object of making the concrete capable of carrying a load which is excessive with the minimum links spaced at the maximum pitch. As an example, the minimum binding may mean the placing of links at a 6-in. pitch but in order to increase the resistance of the concrete the designer may decide to specify links at a 4-in. pitch; if during the execution of the work these links are carelessly placed, and instead of keeping to the 4-in. pitch the spacing is increased to 5 in. or 6 in. in some places and reduced to 2 in. or 3 in. in others, the strength of the column will be affected and the value of the additional links will be lost. During the placing of the concrete the links are sometimes displaced by pushing them downward, with the result that wide gaps occur in the length of column where there are no links at all, and it will be obvious that in this case the concrete at these points will not be hooped and in consequence the normal working stress cannot be developed without risk of failure. The author has noticed this particular defect on several schemes, and there is a general tendency to space the links approximately on account of the impression that they are of secondary importance in the finished work.

These general notes on the points considered by the designer when dealing with the calculations for columns should be sufficient to indicate the complicated nature of his problem and show how full advantage can be taken of the possibilities of the scientific material by arriving at the most economical arrangement of diameter to length, condition at ends, and ratio of binding to concrete; and at the same time it will be apparent that all these refinements and all the care taken in the design are inspired by the faith of the designer in the man on the job, as he calculates a column on the natural assumption that the work will be executed in the best possible manner and absolutely in accordance with the drawings he prepares.

Concrete Core.

The concrete in a reinforced concrete column can be considered as consisting of two parts, (*a*) the core and (*b*) the protective covering. *Fig.* 63 shows the sectional plan of a typical column reinforced with four vertical rods and having an overall size of 15 in. by 15 in. The protective covering must be not less than 1½ in. thick or less than the diameter of the vertical steel rods, and assuming that these rods are not more than 1½ in. diameter, the covering will be 1½ in. thick outside the rods, giving an internal core of 12 in. by 12 in. The covering is neglected in the calculations as regards the ratio of diameter to height

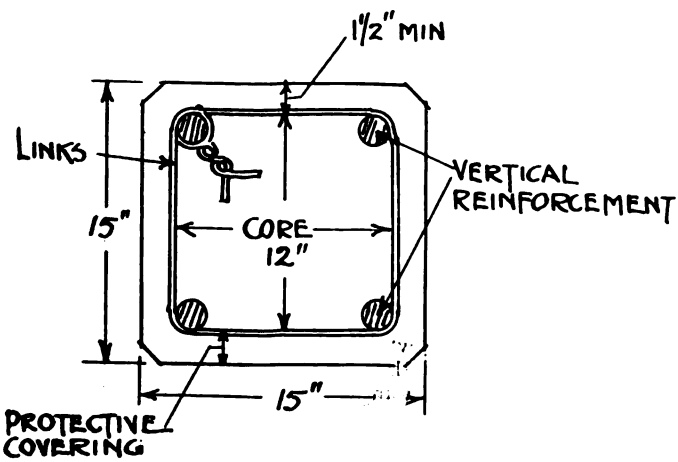

FIG. 63

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and also in the area of concrete to resist the load. The "effective" diameter—which is the diameter of the hooped core—and the area of the concrete within the hooped core only are considered.

In a short column which has the minimum permissible binding the concrete can be calculated to resist safely a compression of 600 lbs. per sq. in., and in the example given in *Fig.* 63 therefore the safe load is 12 in. × 12 in. × 600 lb. = 8,640 lbs., plus the value of the resistance of the steel.

It may be considered that the concrete outside the vertical rods will also be capable of carrying some of the load, but it will be clear that in case of fire this covering is liable to crack and possibly break away; in addition the binding around the core increases the resistance of the concrete as it restrains the material, and it would not be economical to restrict the safe load per sq. in. to the comparatively low limit of the concrete which is not hooped, especially as this would have the effect of reducing the working stress on the steel on compression.

The variations in the working stress on the concrete caused by different ratios of binding are not given here, as this matter is too complicated for these simple notes, but the point which must be emphasised is that the binding does increase the resistance of the concrete, and it is essential that the core of the column be well executed with dense concrete in order to provide the strength when it is actually required.

Vertical and Lateral Reinforcement.

In order to assist the concrete to support the load and avoid the necessity of providing very large columns, reinforcement is added in the form of vertical rods which extend through the full height of the column. These rods are placed as near to the outer surface as possible in order to get the maximum value out of the steel, because the material which is the greatest distance from the axis of the column will be of the most assistance in reducing the flexibility of the member.

When four vertical rods are used they are placed in the four corners of the column, as shown in *Fig.* 63, but six, eight or any number may be adopted according to the requirements. In the case of circular or octagonal columns, where circular binding is used at least six vertical rods must be adopted and frequently eight are provided. As the steel rods are provided to resist compression and assist the concrete, the working stress is limited to fifteen times that on the concrete core, as explained in the notes dealing with elasticity; it is therefore not economical to design the columns with the maximum steel and the minimum concrete because the former cannot be stressed up to its full working strength without injury to the latter. The minimum steel permitted in the L.C.C. Regulations is 1 per cent. of the hooped core, and the diameter of the rods must not be less than ½ in. or greater than 2 in.

The lateral reinforcement consists of spiral binding or separate links taken around the outside of the vertical rods to form a kind of cage, and the spacing of the binding or links must not exceed six-tenths of the effective diameter of the column at any part of the length while at the ends the pitch must not exceed three-tenths of the effective diameter for a length equal to one and a half times the effective diameter. The object of the lateral reinforcement is to prevent the bursting outwards of the concrete and steel under the compression caused by the load. When the load is applied to the column the tendency in the concrete is to spread outward as it becomes shortened, or, in other words, the tendency is to run away and let the load go by; the lateral binding acts as a tie and confines the concrete in a cage from which it cannot escape without bursting the steel links. In addition the compression on the vertical steel rods tends to bend these in a similar manner to that described in connection with the cane walking-stick, as the ratio of the diameter of the rod to its length makes it very flexible, and the lateral binding acts as a tie and holds the rod back against the concrete. The steel reinforcement can obviously only bend outward on account of the mass of concrete which is behind each rod, and without the lateral binding it would be likely to spring out under the load and burst through the comparatively thin protective covering.

All joints between vertical rods which

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extend through several stories must be well executed because the load on the upper rod must be transmitted directly to the lower one and not be left in a manner which results in the load being passed on to the concrete. Also, at the foot of the vertical rods the ends must be turned out to avoid the load on the steel being concentrated on a very small area of concrete equal to the sectional area of the rods, as this will theoretically result in the concrete being overloaded by fourteen times the working stress and there will be a tendency for the rods to be pushed through the concrete on account of the latter being crushed by the excessive pressure.

Junction with Beams.

It is very important that the junction of the column with the beams which it is supporting is properly executed, as it is necessary for the loads to be transmitted from the beams to the column over the whole area of contact, and in addition the fixing of the column must be provided for as previously mentioned. Owing to the stoppage of work that usually takes place at the head of the columns there is a serious risk of defective work at the junction unless considerable care is exercised.

The columns must also be kept perfectly plumb and upright during construction, otherwise they will be subject to conditions and stresses which have not been provided for in the design. If a column is leaning over in one direction and a vertical downward load is applied at the top the tendency is to push the column still further over in the direction in which it is leaning and the simple condition of direct compression is not attained. A tendency to push the column over sideways means that it is being forced to act as a beam or cantilever, and instead of the whole area being under compression one face or edge will be called upon to resist a tensional stress.

In many cases the designer has to deal with eccentric loads, as when a bracket is attached to one face of a column; in such cases provision is made for the stresses caused by the load not being central on the top of the column, but it is a serious matter if eccentric loading is caused in a column when no special provision has been made for it. It should be realised that the execution of the columns in a reinforced concrete structure demands the greatest possible accuracy and care, because these members are the most important in the whole building. This does not mean that care is of secondary importance when dealing with beams and slabs, but the failure of a beam or slab may result in a local failure only whereas the failure of a column may result in the failure of the greater part or the whole of a building and in this sense it is vital to the safety of the structure that the columns are well and truly executed.

(*To be continued.*)

Repairing Large Concrete Pipes.

In the repair of a number of defective joints in a concrete pipe-line in British Columbia, the following method was used with success by Mr. F. M. Preston, City Engineer and Water Commissioner of Victoria, B.C. Twenty-four-ounce copper strip was rolled to fit the circumference of the pipe and extend two-thirds of the distance around it, or practically to the high-water level in the pipe. This strip is 4¾ in. wide, and has a V-shape depression ½ in. deep and ¾ in. wide formed longitudinally along its centre line; it is attached to the pipe with the "V" projecting into a small groove chipped into the joint and clamped in place by two 2½-in. round steel rods and turnbuckles. Packing between the copper strip and the pipe is provided by two 1½-in. strips of sheet lead. While the steel clamps are being adjusted the band is held in place by a small wire rope passed around the copper and tightened with a simple form of clamp. The pipe-line in question is 27½ miles in length, and several hundreds of these flexible joints were used to repair defects in the lock-joints between the pipes. The pipes were 3 in. thick, with an internal diameter of 42 in. After three years' service it is found that this method of repair is quite effective and sufficiently flexible to allow for expansion and contraction in the concrete owing to temperature changes in a district where the thermometer registers as low as 18 deg. in the winter.

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SWANAGE SEA DEFENCES.

A PORTION of the sea-wall at Swanage, measuring 250 yds., was damaged by a heavy sea towards the end of 1923, and it was found necessary to underpin the wall throughout the damaged length and reconstruct the portion which had collapsed. The piling and boarding being accomplished, the concrete underpinning was carried out about 4 ft. beyond the face of the wall. The concrete for this consisted of Portland cement, sand and broken scars in the proportion of 1 : 2 : 4, and was carried down by a toe piece into the clay about 12 in. The scars are pieces of stone chipped from blocks at the quarry head by the banker masons when fashioning the stone.

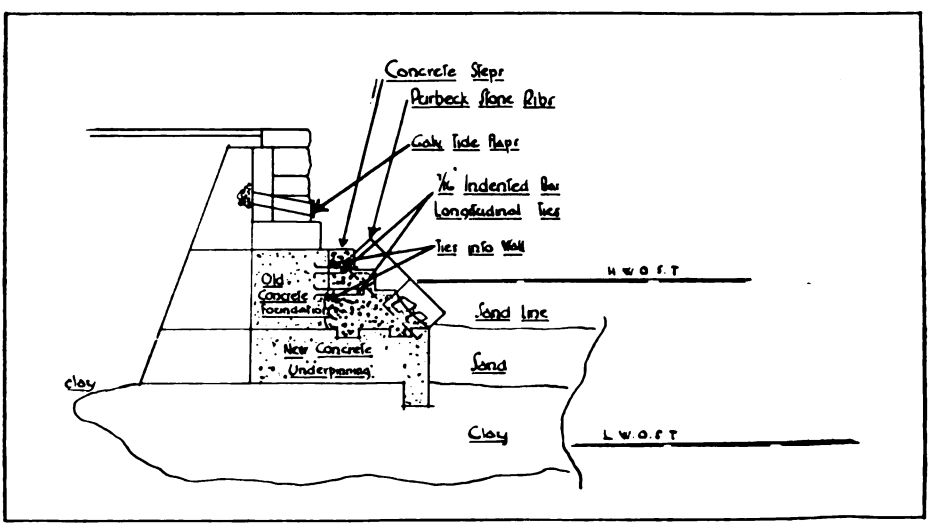
The concrete steps, superimposed on the underpinning, were cast in alternate bays of 9 ft. lengths in the specially made wooden moulds, so that a groove was left in the concrete about 12 in. wide to take the Purbeck stone ribs, which were inserted every 9 ft. to break the waves which rush with great force round the bay.

On the ends of the mould two blocks were fixed, which, when the moulds were removed, left a mortise in the end of the bays forming a key for the other bays.

The core of the steps consisted of fairly large stones which were placed in the 9-ft. mould, in irregular formation on cement mortar, which formed a rough concrete walled core, packed into position inside the mould which was shaped to the required section. The concrete core was constructed of a mixture of about 1 : 2 : 5. To present a hard surface to the impact of the waves the concrete core was faced with 2 in. of concrete composed of 1 part of Portland cement to 2 parts of granite chippings $\frac{5}{8}$ in. to dust.

During the construction of the steps, two longitudinal ties of $\frac{3}{8}$ in. square twisted steel bar were placed in position throughout the length of the concrete steps, and tied into the old concrete foundation with cross-ties every $4\frac{1}{2}$ ft. By this means the whole of the bays were connected to each other and to the foundation of the old wall. The steps were continually washed by the sea during their construction.

The stone steps leading down to the sand from the esplanade were returned on the front with reinforced concrete steps, the ends of the steel bars being turned over and concreted into mortises cut out of the end section of the stone steps. The concrete platform at the bottom of the steps was reinforced with $\frac{3}{8}$ in. square steel twisted bars, placed about 6 in. inside the outer edge of the



SWANAGE SEA DEFENCES.

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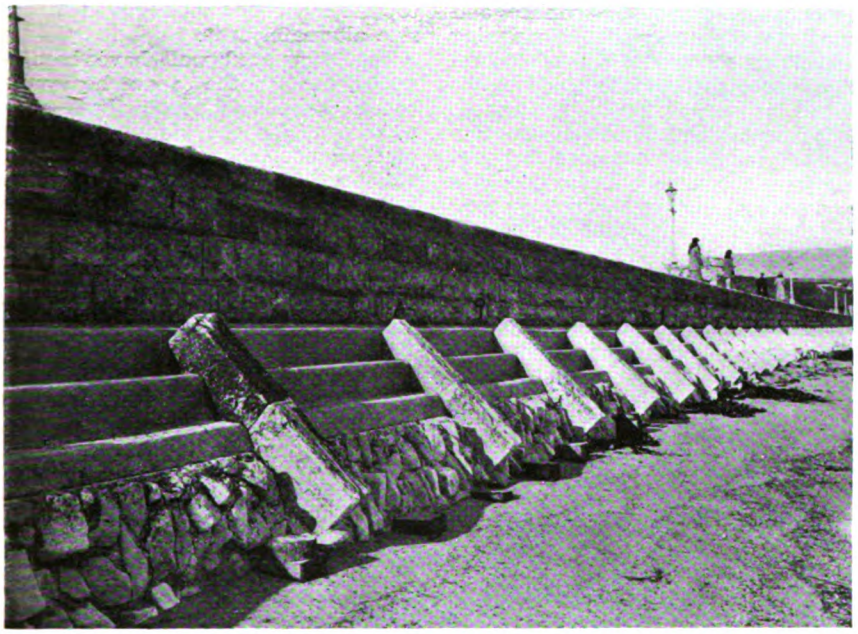
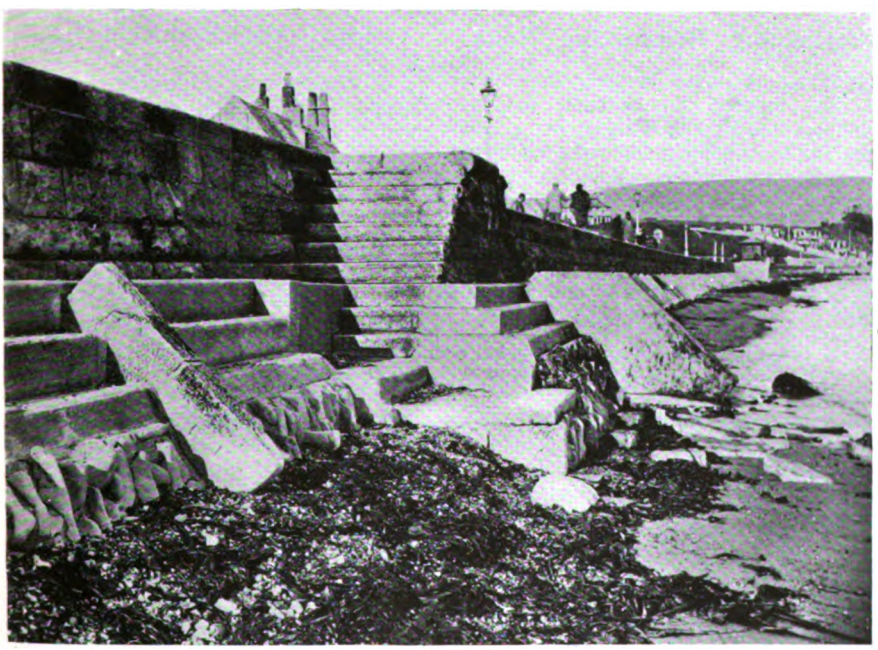


General View.



Reinforced Concrete Rib shown Projecting on Right of Steps.

SWANAGE SEA DEFENCES. (*See p.* 408.)

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platform 2 in. below the surface and anchored to the concrete underpinning with steel ties. The ends of these boundary bars were attached to the longitudinal ties.

An interesting feature in the construction is the reinforced concrete rib, projecting from the steps. It was cast in a 12-in. groove in the steps with two steel bars placed about 2 in. from the surface of the rib, looped over the top longitudinal tie, passing round the perimeter, and finally looped to the bottom longitudinal tie passing through the steps. Four diagonal ties were looped over the top bar to the bottom bar of the opposite side alternately.

The steps and wall have been well tested during the very heavy seas, which have continually buffeted the walls, sending the spray about 40 ft. high, and have emerged from the trial without a flaw, although the old part has been badly shaken.

The Surveyor to the Swanage Urban District Council, under whose direction the work has been carried out by direct labour is Mr. C. G. J. Ratcliff, A.M.I.S.E., who is also responsible for the design of the reinforced concrete rib and steps.

Questions and Answers relating to Reinforced Concrete.

Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.—ED.

Lengthening Reinforcing Rods.

QUESTION.—What in your opinion is the best method (apart from welding, which cannot be relied upon in my district) for lengthening the rods for spans of 30 ft. and over ?—FERRO.

ANSWER.—The best way to lengthen rods for spans of 30 ft. and over is to lap the rods for a length of 48 diameters and provide a hook at the end of each rod.

Cement Wash.

QUESTION.—Will you kindly inform us if you know of any simple and reliable means of adding anything to cement in order to fasten it to the wall and prevent it from washing or brushing off when it is applied as a wash with a white-wash brush ?—J. D.

ANSWER.—The following are recommended to make a good cement wash which will resist the damp and will not crack and chip off : (1) Spread Portland cement dry on a floor for a period of 6 hours, later mixing with water in a large tub. Consistency to be judged by the condition of the wall to which it is to be applied. Add 1 quart of soluble glass to each 5 gall. ; stir well when using. (2) 50 parts of Portland cement to be mixed into a wash. Then mix 5 parts each of plaster of Paris and lime into another wash, adding to the cement, and stir well. 5 parts of powdered glue size should be melted with boiling water and poured into the wash, then stir well, after that melt a little alum and stir in.

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Thames ballast	,,	10	6
Broken brick (¾ in.)	,,	11	6
Best British Portland Cement	per ton 58s. to	63	0
"Ferrocrete" Rapid-Hardening Portland Cement	delivered London 10s. per ton extra		
"Super-Cement"	per ton 88s. to	93	0
"Lightning" Brand Aluminous Cement	,, 100/- f.o.r. works.		
Ciment Fondu	130/- ex stores		

BOARDING FOR SHUTTERING—		Sawn. s.	Sawn. d.	Wrot. s.	Wrot. d.
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in. . . . . from £23 per standard
3 in. by 6 in. and 3 in. by 7 in. . . . . ,, £23 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	0
9/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds	,,	14	6
¼ in. Rounds	,,	15	6

Breeze Slabs per yd. super: 2 in., 1/11; 2½ in., 2/4; 3 in., 2/9; 4 in., 3/6.

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 : 2 : 4—		s.	d.
Do. do. in foundation	per yard cube	48	6
Do. do. in columns	,, ,,	53	6
Do. do. in beams	,, ,,	53	6
Do. do. in floor slabs 4 in. thick	per yard super	5	8
Do. do. in floor slabs 6 in. thick	,, ,,	8	5
Do. do. in floor slabs 9 in. thick	,, ,,	12	3
Do. do. in walls 6 in. thick	,, ,,	8	8

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—		s.	d.
From ¼ in. to ⅝ in.	per cwt.	25	0
,, 9/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2⅞ in.	,,	23	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1¼*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2¼*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.* (per bend per cwt.).

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.* (per bend per cwt.).

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	,, ,,	1	1½
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1½
Do. do. as last, in narrow widths	,, ,,	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Carpenters working on old shuttering, 1/10½; Labourers on building works, 1/4½; Men on mixers and hoists, 1/5½; Bar-benders, 1/5½.

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Prospective New Concrete Work.

ABERTILLERY.—*Houses.*—The U.D.C. proposes to erect 100 houses.

ACCRINGTON.—*Houses.*—The Corporation is negotiating for the purchase of 13½ acres of land on which to build 160 houses.

ALDERSHOT.—*Houses.*—The B.C. proposes to build 100 houses by direct labour.

AYSGARTH.—*Bridge.*—The R.D.C. has reminded the County Council of the need for a new Thwaite bridge.

BARNES (LONDON).—*Bridge.*—Proposals for relief works include the construction of a bridge over Beverley Brook.

BELFAST.—*Houses.*—A scheme for the construction of 500 houses is to be considered by the Corporation.

BILLINGHAM.—*Houses.*—The U.D.C. has applied for sanction to erect 66 houses on the Chilton Lane site.

BLACKWATER.—*Footbridge.*—The Romsey R.D.C. is considering the erection of a footbridge at Blackwater.

BLYTH.—*Reservoir.*—The T.C. has a scheme in hand for the construction of a reservoir at Horton, estimated to cost £30,000.

BRADFORD. — *Houses.* — It has been decided to erect 74 houses at Bradford Moor.

BREDBURY.—*Footbridge.*—A footbridge is to be constructed at Bredbury Station.

BURTON-ON-TRENT. — *Road.* — Plans have been prepared by the Borough Surveyor for the construction of a road from Wellington Street to Branstone, which is estimated to cost £40,000.

CHESTER-LE-STREET. — *Houses.* — The Chester-le-Street R.D.C. has applied to the M.H. for sanction to borrow £98,000 for the erection of 100 houses at Fatfield, 60 houses at Lumley, and 70 houses at Edmondsley.

CHICHESTER.—*Houses.*—The City Council has applied for permission to borrow £22,000 for the construction of 50 houses in Lewis Road.

DORCHESTER.—*Bridges.*—The Dorset C.C. has applied for permission to borrow £9,128 for the erection of bridges.

DUNMOW.—*Houses.*—The R.D.C. has a scheme in hand for the supply of 100 houses.

EAST HAGBOURNE.—*Bridge.*—The Wallingford T.C. has decided to erect a bridge at East Hagbourne.

GLOUCESTER. — *Houses.* — The City Council has approached the M.H. for permission to erect 50 houses.

GOOLE.—*Houses.*—Arrangements have been made for the purchase of land on which to erect 200 houses.

HASTINGS.—*Pier.*—Improvements to the pier are to be carried out at a cost of £30,000.

HEMSWORTH.—*Houses.*—The R.D.C. has applied for sanction to a loan of £53,684 for the erection of houses.

HEVENSHAM.—*Road.*—A road from Hevensham to the old toll-bar at Levens is to be constructed at a cost of £29,760.

HOUGHTON-LE-SPRING. — *Houses.* — Sanction to a loan of £25,584 has been applied for by the R.D.C. for the construction of 62 houses.

IPSWICH.—*Concrete Factory.*—A suggestion that the extensions to Messrs. Harris's bacon factory at Ipswich be erected in concrete is being considered.

IRTHLINGBOROUGH.—*Bridge.*—A bridge is to be erected over the river and railway at a cost of £50,000.

KIRKBY STEPHEN. — *Reservoir.* — The East Westmorland R.D.C. proposes to carry out improvements to the water supply of Kirkby Stephen, which includes the construction of a 60,000-gallon reservoir.

LITTLE CROOK.—*Bridge.*—It is proposed to erect a bridge at Little Crook, near Alton.

LOTHIAN.—*Concrete Bridges.*—Concrete bridges are to be erected over the Garvald stream at a cost of £350.

LOUGHBOROUGH.—*Concrete Bridge.*—The Corporation has applied for a grant towards the cost of reconstructing the existing bridge over the canal in Meadows Lane in reinforced concrete.

MANCHESTER.—*Houses.*—The Corporation proposes to purchase 45 acres of land in the Moston district on which to erect 500 houses.

NEWCASTLE-ON-TYNE. — *Road.* — The Finance Committee is considering a proposal for the construction of a road from City Road to Barras Bridge, which is estimated to cost about £1,000,000.

NOTTINGHAM.—*Concrete Houses.*—The Corporation Housing Committee proposes to erect 1,000 "Crane" type concrete houses.

PAIGNTON.—*Houses.*—The U.D.C. proposes to erect 100 houses.

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PERTH.—*Houses.*—The T.C. proposes to erect 150 houses.

PORTSMOUTH.—*Road.*—The Corporation has applied for a loan of £30,000 for the purchase of land in connection with the construction of a proposed road at North End.

RAWMARSH.—*Swimming Pool.*—A proposal for the construction of a reinforced concrete swimming pool 80 ft. long by 35 ft. wide is being considered by the Central Miners' Welfare Committee and the Rawmarsh U.D.C.

ROCHDALE. — *Reservoir.* — The Guardians propose to reconstruct the floor of the reservoir at Birch Hill with reinforced concrete, which is estimated to cost £2,000.

RUSHDEN.—*Houses.*—The U.D.C. has applied for sanction to a loan of £18,500 for the construction of 38 houses.

SCARBOROUGH.—*Sea Defence Works.*—The T.C. has asked the M.H. for sanction to borrow £26,000 for sea defence works.

SEAHAM HARBOUR. — *Houses.* — The U.D.C. proposes to build 300 houses.

SIDMOUTH.—*Sea Defence Works.*—A loan of £5,099 has been applied for by the U.D.C. for sea defence works.

SOUTH SHIELDS.—*Road.*—The T.C. proposes to spend £120,534 on the extension of King George Road.

SOUTHPORT.—*Houses.*—The T.C. has applied for sanction to build 283 houses.

SUNBURY.—*Bridge.*—A bridge is to be erected to connect up Rivermead Island, which has been purchased by the Sunbury U.D.C. for a public pleasure ground.

SWANSEA.—*Bridge.*—Between £30,000 and £40,000 is to be spent on the construction of a bridge over the King's Dock level crossing.

THORNABY. — *Houses.* — A loan of £40,000 has been applied for by the T.C for the construction of 100 houses.

Tenders Accepted.

ABERCYNON.—*Reservoir.*—The U.D.C. has accepted the tender of Mr. John Sutherland, of Abercynon, at £8,353 16*s.*, for the construction of a mass concrete reservoir.

AXBRIDGE. — *Concrete Houses.* — The R.D.C. has accepted the tender of Mr. J. Moore, of Nailsea, for the erection of six concrete houses.

BARNSLEY.—*Concrete Bridge.*—The T.C. has accepted the tender of Messrs. Wellerman Bros., Sheffield, at £6,274 10*s.*, for the construction of a reinforced concrete bridge.

BLACKBURN.—*Concrete Bridge.*—The T.C. has accepted the tender of Messrs. Grays Ferro-Concrete Co., Ltd., of Glasgow, for the construction of a reinforced concrete bridge over the L. M. & S. Railway.

BLAIRGOWRIE.—*Concrete Tanks, etc.*—The T.C. has accepted the tender of Messrs. Andrew MacCracken & Sons, Motherwell, for the construction of concrete tanks, etc.

CHURCH STRETTON. — *Bridge.* — The R.D.C. has accepted the tender of Mr. J. Bromley, Church Stretton, at £1,182, for the construction of a reinforced concrete bridge over the Longnor Brook.

EVESHAM.—*Bridge.*—The Worcestershire C.C. has accepted the tender of Messrs. T. Vale & Sons, Ltd., Stourport, at £27,941 19*s.* 8*d.*, for the construction of a reinforced concrete bridge over the River Avon.

HULL.—*Concrete Houses.*—The Corporation has accepted the tender of Mr. F. Bilton, Hull, for the construction of 200 concrete houses.

LEEDS.—*Sewage Works.*—The T.C. has accepted the tender of Messrs. H. Atkinson & Sons, Ltd., at £12,386 1*s.*, for the construction of 92 concrete cylinder sections, and for sinking 50 existing concrete cylinder units for the foundations of the sewage tanks at Knostrop.

LYDNEY. — *Concrete Reservoir.* — The R.D.C. has accepted the tender of Messrs. W. T. Nicholls, Ltd., Gloucester, at £2,097, for the construction of a reinforced concrete reservoir and addition to the pumping station at Ferneyley Wood.

PARKGATE (YORKS).—*Reservoir.*—The Rawmarsh U.D.C. has accepted the tender of Mr. A. Graham, Fartown Green Road, Huddersfield, at £9,128, which includes the construction of a 500,000-gallon reinforced concrete reservoir at Monkwood and a 60,000-gallon reinforced concrete reservoir at Upper Haugh.

SUTTON COLDFIELD. — *Bridge.* — The T.C. has accepted the tender of the Reinforced Concrete Construction Co., Ltd., of Old Trafford, at £1,031 12*s.* 7*d.*, for the construction of a reinforced concrete bridge over Plants Brook in Penns Lane.

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Recent Patent Applications.

206,827.—C. Zeiss: Reinforced concrete domes.
212,268.—A. N. P. Jacobs: Casting moulded articles from magnesia cement.
223,595.—Henderson Corporation: Reinforced-blockwork columns, girders, and beams.
226,837.—J. M. Blasco: Drums applicable to concrete mixers.
229,362.—W. Bagguley and R. Hickton: Manufacturers of bricks or blocks.
229,364.—C. Boot: Buildings.
229,368.—E. W. Savory: Slab-building construction.
229,743.—J. S. Baines: Methods of building construction.
230,678.—H. O. Keay: Process of cementing pulp stones.
231,007.—A. Gaertner: Cement and cementitious mortar.
231,242.—V. Lefebure: Cementitious materials.
231,308.—A. Wust: Manufacture of building-blocks.
231,376.—A. E. Holmes: Shuttering.
231,644.—A. E. Cudmore: Concrete walls.
231,702.—Deutsche Patent Ankerschienen Ges Halfen, and J. Kramer: Channel-shaped anchor-rails for ferro-concrete beams and girders.
232,155. Soc. Anon. Des Ciments Français, and Bureau d'Organisation Economique: Manufacture of cement.
232,467.—H. Siegwart: Manufacture of tubular bodies from concrete.
232,761.—A. E. Vidal (Kraftbau Patent Verwertungs Ges): Apparatus for spraying cement.
232,898.—L. G. Patronilleau and Alumine et Derives: Manufacture of aluminous cements.
233,001.—E. B. Hack and E. J. Burt: Manufacture of concrete.
233,105.—B. C. Barton: Reinforced concrete walls.
233,230.—H. Freeman and F. P. Bloomfield: Floors, roofs, and ceilings, and hollow reinforced concrete girders therefor.
233,386.—G. K. Green: Building concrete *in situ*.
233,455.—P. Davies: Concrete structures.
233,560.—L. A. Pitcher: Building blocks.
233,623.—J. Bosswell: Concrete structures.
233,795.—F. F. Mote: Shuttering.
233,819.—A. Brune: Concrete tiles.

New Companies Registered.

CEMENT-GUN CO. (GREAT BRITAIN), LTD. (205115). Registered April 7. Manufacturers of and dealers in cement guns. Nominal capital, £1,000 in £1 shares. Directors: To be appointed by subscribers. Subscribers: M. Whitaker, 2, Rudall Crescent, N.W.3; E. Reckling, 100, Victoria Street, S.W.1.

EPSOM LIME WORKS, LTD. (205095). Registered April 6. The Lime Works, College Road, Epsom. Manufacturers of concrete, brick, tiles, pipes, etc. Nominal capital, £1,000 in £1 shares. Permanent directors: A. P. Crosby, Shafields, Cranley Road, Guildford; G. B. Hindmarsh, 8, Alexandra Road, Epsom.

EVANS BROS. (CONCRETE), LTD. (206541). Registered June 10, 1925. Greenhill Lane, Riddings, Alfreton, Derbyshire. Erectors of concrete buildings. Nominal capital, £7,000. Directors: L. P. Evans, The Grove, Swanwick, near Alfreton; J. L. Llewellyn, Monument Hill, Codnor Park, near Alfreton.

Change of Address.

MESSRS. TOPPING & LEGGATT, Contractors, have removed to 11 Victoria Street, Westminster, S.W.1. (Telephone: Victoria 5747; Telegrams: Smokechim, Parl, London.)

THE London office of Messrs. Ransome Machinery Co. (1920), Ltd., has been changed to 42–50 Victoria Street, Westminster, S.W.1. (Telephone, Victoria 8060; Telegraphic Address, Ransomaco, Sowest, London.)

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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EDITORIAL NOTES.

Tests of Cement for Expansion.

WHEN cement is tested for soundness by the Le Chatelier apparatus the cement paste gauged with water is filled into a small split cylinder which is immediately immersed in water. After twenty-four hours the cylinder with the cement block it contains is boiled for six hours and the expansion of the block registered by the movement of two pointers attached to the mould. In addition to this expansion in boiling water, which is the basis of the tests as embodied in the British Standard Specification, there is frequently an expansion occurring during the first twenty-four hours' immersion in cold water when the cement is setting and hardening, and suggestions have been made from time to time that this cold water expansion registered by the Le Chatelier apparatus should be regarded as an indication of the expansive properties of the cement when used in constructional work. Those with experience in cement testing have generally been able to connect the cold water expansion with other circumstances, such as the condition of the apparatus

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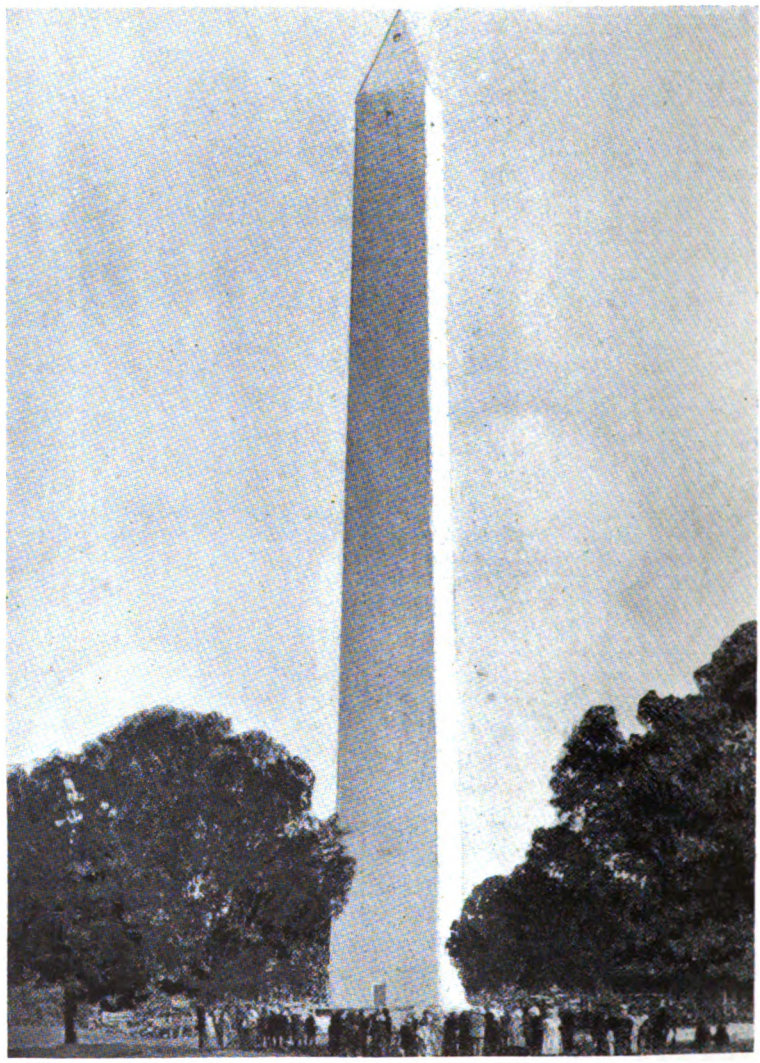
and the setting properties of the cement, and have deprecated any attempt to draw from the results conclusions that were not warranted.

It is interesting to learn from the report of the National Physical Laboratory for the year 1924 that this subject has been under investigation at the instigation of H.M. Office of Works, and the conclusion has been reached that measurements of the cold water expansion cannot give any information as to the expansion to be allowed for in constructional work. The thoroughness and impartiality with



The Jefferson Davis Monument, Fairview, U.S.A.

[This monument is 351 ft. high (the second highest in the United States), and is built entirely of concrete. The base is 35 ft. sq. The concrete was poured in moving forms, and hoisted direct in the tip-carts which brought the material from the mixer.]

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which scientific investigations are conducted at the National Physical Laboratory make it certain that this is a conclusion to be accepted without dispute, and there should be no scope for further controversy upon the matter.

Sodium Silicate as a Concrete Improver.

SODIUM silicate has frequently been referred to in the pages of this journal as a means of hardening concrete, and two features have been emphasised. First, sodium silicate is a material of known composition which can be bought from more than one chemical manufacturer; and, second, its action upon concrete is technically sound because it leads to the conversion of lime in the set cement to the more resistant silicates of lime. The improving qualities of sodium silicate are well illustrated by tests carried out by the National Physical Laboratory and published in the report for 1924. The tests were made by the abrasion of cylinders of cement mortar upon a steel disc loaded with sand, and the amount of wear of a cylinder per 1000 revolutions of the disc is an index of the resistance to abrasion. The results are as follows:—

	Resistance to Abrasion (Wear of cylinder in in. per 1,000 revolutions of disc).	
	Air Cured	Water Cured
Cement and sand mixture	3·95	0·64
Cement, sand and silicate of soda mixture	1·87	0·47
Cement and sand cylinder painted with silicate of soda	1·75	0·50

These figures illustrate the great value of curing concrete under moist conditions and also show that a water-cured concrete is rendered 25 per cent. more resistant to abrasion by the use of sodium silicate.

Calcium Chloride and Concrete.

ACCORDING to the results of a series of tests undertaken by Professor Duff Abrams, and summarised in this issue, the problem of the reduction in the rate of hardening of concrete as the temperature decreases is shown to be assisted by the addition of a small amount of calcium chloride to the gauging water. It is well-known that the addition of any soluble salt to water lowers the freezing point, but calcium chloride, according to these tests, possesses peculiarly suitable properties for concrete. In the first place only a very small amount is required, being from 2 per cent. to 3 per cent. of the weight of the cement used, whilst the ultimate strength of the concrete is not affected. The action of calcium chloride in increasing the rapidity of hardening of concrete is seen to have a maximum improving effect of 70 per cent., but this figure applies only to certain mixtures, and at an age of two days. With other mixtures and at longer dates the degree of improvement is notably less.

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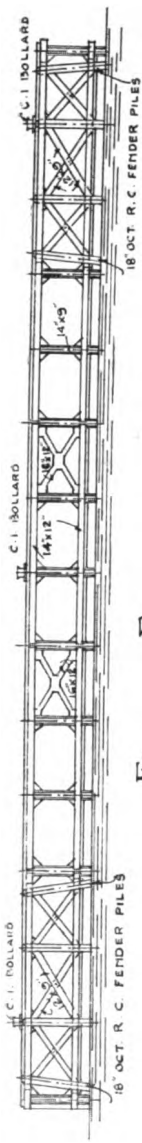
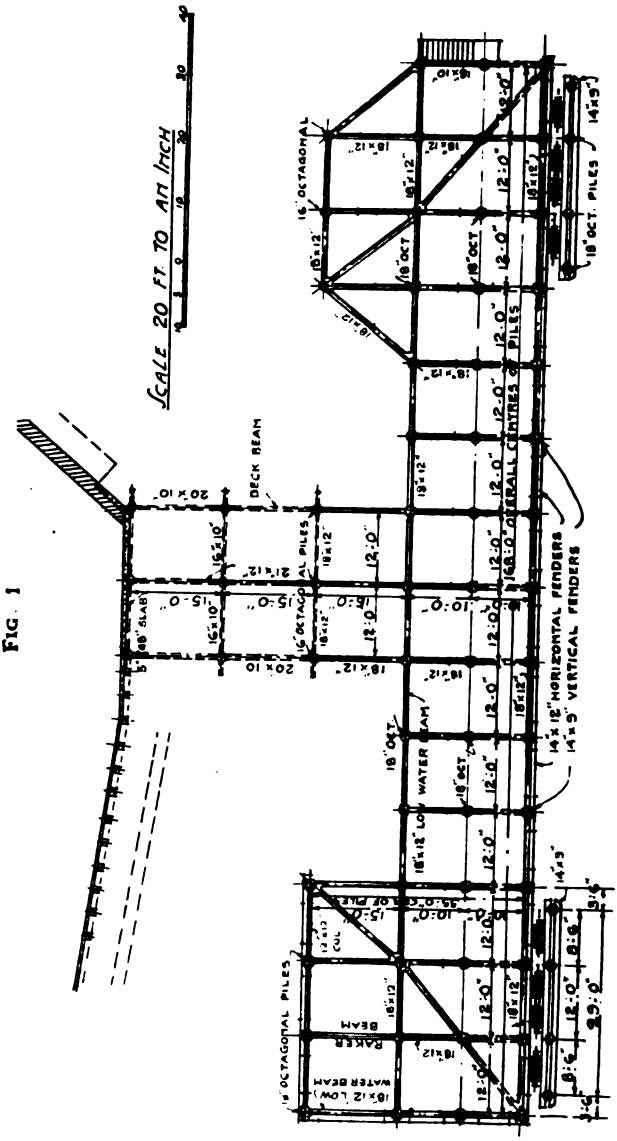


FRONT ELEVATION

FIG. 1



LOW WATER BRACING PLAN

FIG. 2.

Reinforced Concrete Wharves in Singapore. (*See p.* 419.)

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Reinforced Concrete Wharves in Singapore.

By A. G. Stredwick, M.I.Struct.E.

MANY reinforced concrete wharves have been built in Singapore since 1910, and the knowledge and experience gained in building and using these wharves is very instructive to those interested in this type of wharf construction.

A wharf has recently been constructed at Pul e Bukom, an island near Singapore, which has incorporated in it the improvements made during this period. From *Fig.* 2 it will be noted that the wharf is built in the form of a "T" with the head of the "T" 168 ft. long and the arm projecting 45 ft. from the shore. The wharf is built on the pile system, the piles being 16-in. and 18-in. octagonal in shape, varying from 40 ft. to 60 ft. in length, while the decking is 7-in. thick, supported by very heavy beams; large knee brackets have been made from all beams and bracings to the columns.

A feature of the wharf is the bracing and the formation of the sea-bed level, which lends itself to this type of bracing. It will be noted from the cross-section of the wharf shown in *Figs.* 4, 5, and 6 that the first two rows of piles are almost of the same length below the bracing to sea-bed, and that the third and fourth rows are of shorter lengths. This system gives the wharf a certain amount of flexibility before the two back rows of piles take up more rigidly the stress incurred by steamers berthing, and the arrangement of bracing cuts the low water work down by half.

The wharf has been designed to accommodate steamers varying from 2,000 up to 15,000 tons, and the depth of water at the head of the wharf at low tide is nowhere less than 28 ft.

In front of the wharf two systems of reinforced concrete fender piles have been driven, as shown in *Fig.* 7. Rattan fenders, which are compressible, have been inserted between the fender piles and the wharf itself, and these reduce the initial shock to the wharf. The rest of the face of the wharf is protected by ballow timber fenders (local hard wood). On the right-hand side of the "T" head facing the sea the back of the wharf is used to accommodate 70 to 100 ton tongkangs (barges) and a 2-ton steam crane has been erected here to assist in loading and unloading them.

To moor large steamers three reinforced concrete dolphins have been erected in the sea, as shown in *Fig.* 8. These dolphins are 12 ft. square and comprise four 16-in. octagonal reinforced concrete piles driven into the sea-bed and reaching up to deck level. They are approachable at all states of the tide by the row-boat bringing in the mooring ropes from a steamer berthing. Six-inch slab walls were built between them, and the area thus enclosed was filled in with sand on a coral foundation. This type of mooring dolphin has given most satisfactory results in practice.

Plain bar reinforcement was used throughout the work and the piles have spiral hooping.

The piles were cast in steel sheet moulds and were allowed to season for two months in the sea before being driven from a floating pontoon. They were driven to a set of ¼-in. for the last five blows from a 2-ton steamer-hammer, and the penetration varied from 22 ft. to 28 ft.

All concrete poured above low tide level was mixed in the proportion of 1 part cement, 1¾ parts sand, and 3¼ parts ¾-in. granite. A cover of 2 in. was insisted upon outside stirrups in all members.

The work was designed by the writer (Engineering Department, Asiatic Petroleum Co. (S.S.), Ltd., Singapore) and carried out by Messrs. United Engineers of Singapore, for whom Mr. Dale, M.I.S.E., acted.

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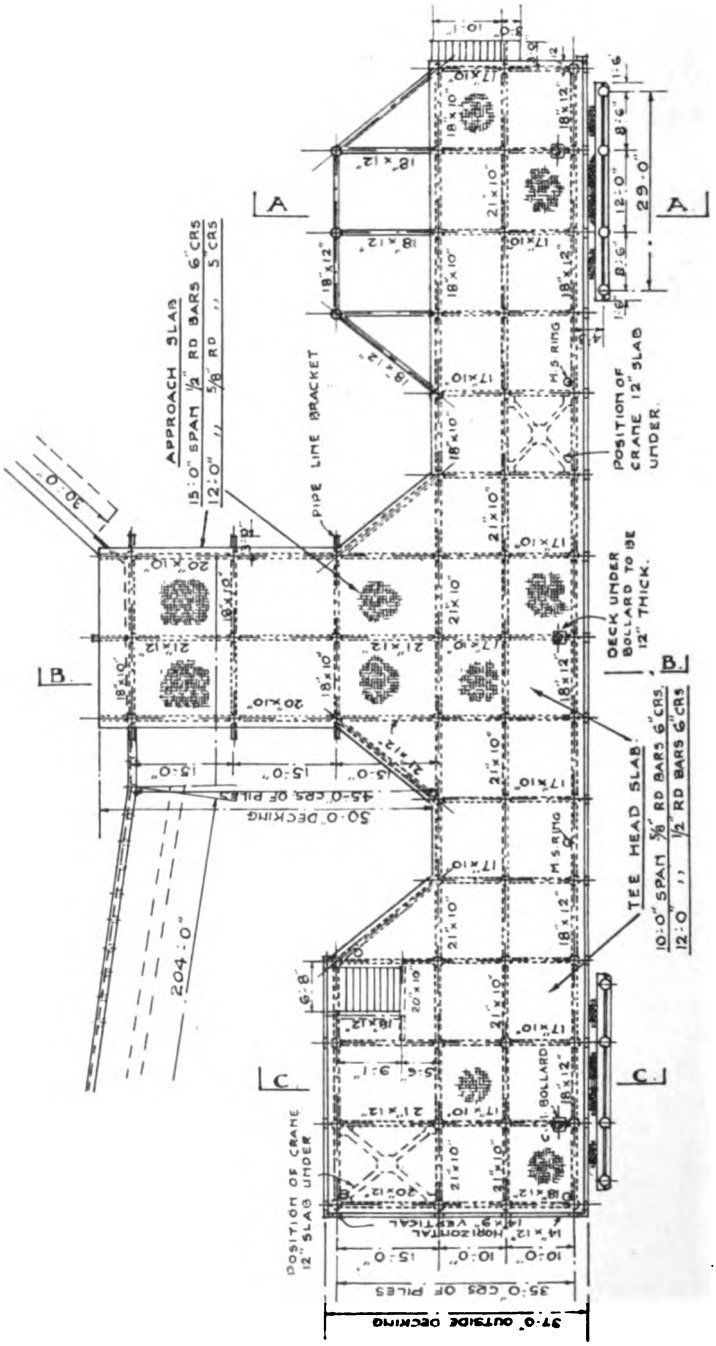

FIG. 3.

Reinforced Concrete Wharves in Singapore. (*See p.* 419.)

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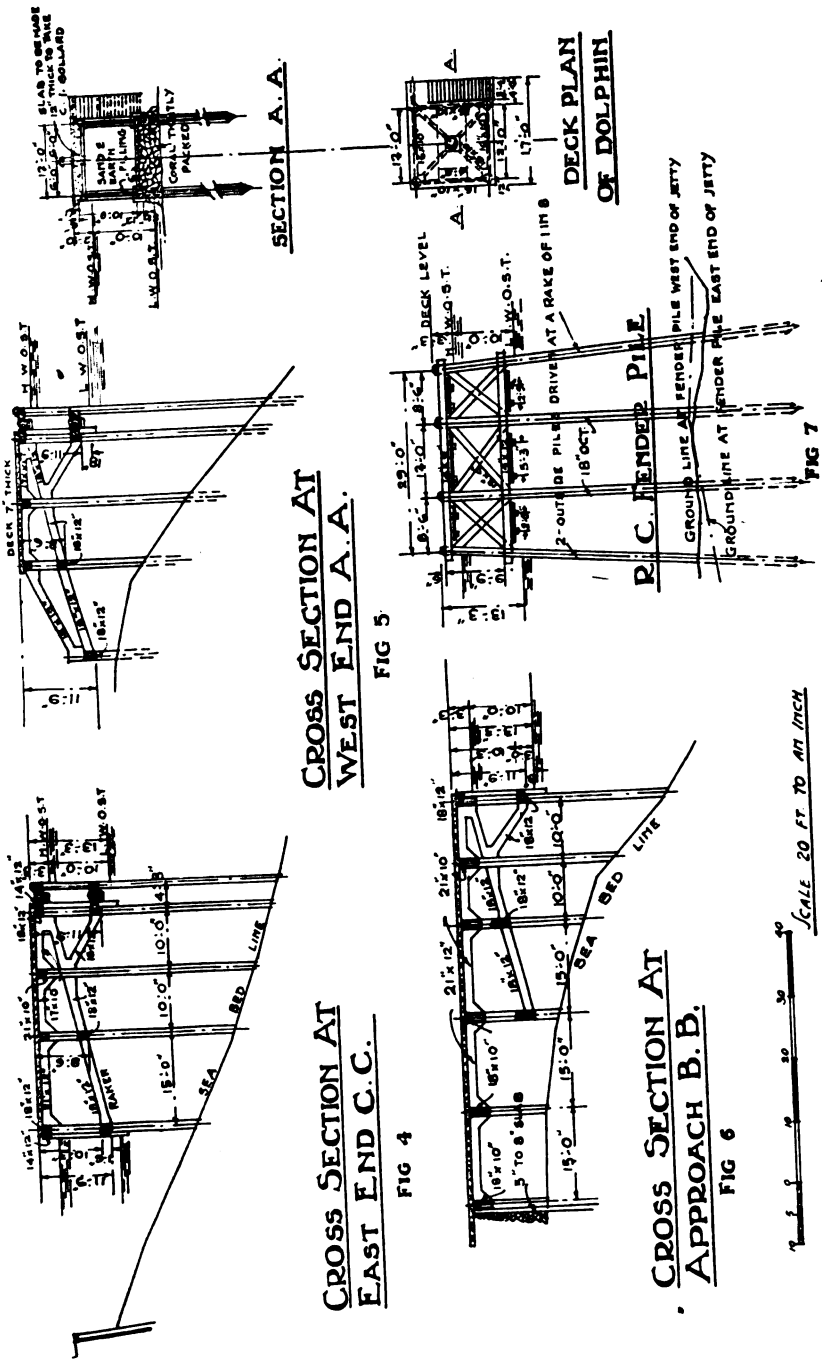

Reinforced Concrete Wharves in Singapore. (See p. 419.)

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Graphical Method of Determining the Spacing of Bent-up Bars and Stirrups in Reinforced Concrete Beams.

By T. H. P. Veal, B.Sc., A.M.Inst.C.E.

THE method outlined may be used in determining the spacing of bent-up bars and stirrups in reinforced concrete beams of the type commonly met with in the design of floor systems. Such beams are usually wholly or partly continuous and the loading is uniformly distributed. The method is, however, applicable to all cases of uniform loading, whether the beam is simply supported or continuous. Many text-books give a similar method for the special case when stirrups of uniform section are used, but they usually omit the general case for varying sections of diagonal tension reinforcement.

In a uniformly loaded beam the shear stress varies from a maximum value (q) at the support to 0 at the centre of the span, and may be represented by a triangle PQR (*Fig.* 1).

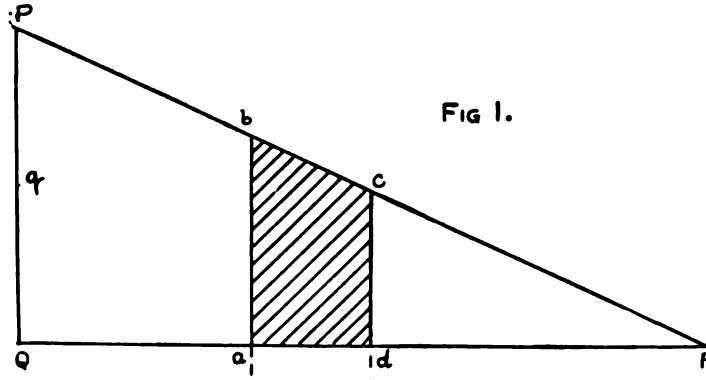

FIG 1.

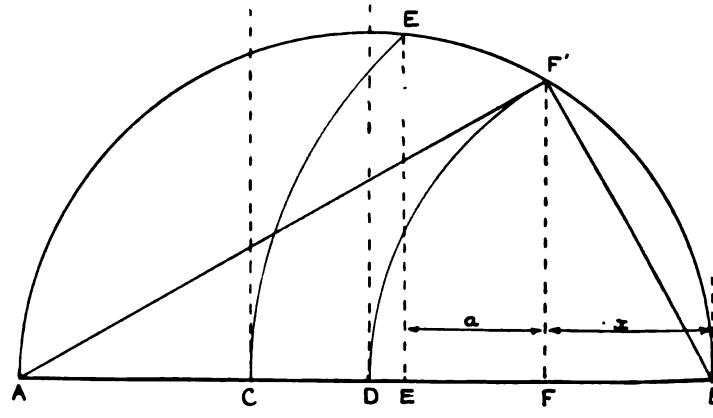

FIG 2.

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The stresses caused by diagonal tension may be assumed proportional to the shear stresses represented by the triangle PQR.

Draw a line AB (*Fig.* 3) to a convenient scale equal in length to the half-span of the beam.

From B, the centre point of the span, draw BC making an angle of about 30° with AB, and let its length represent to some scale the shear force given by the area of the triangle PQR (*Fig.* 1).

If L is the half-span in inches
,, b ,, ,, breadth of the beam in inches
,, q ,, ,, shear stress at A in lbs. sq. inch.

the shear force represented by $PQR = \frac{1}{2}\, q.b.L.$

Suppose six bars are used to resist the stresses due to the bending moment at the centre of the span, and that four of these bars are to be bent up to help in resisting the negative bending moment at the support and part of the diagonal tension. Let the area of the bars to be bent up be A_1, A_2, A_3, A_4 respectively and let A_s be the area of each of the stirrups required in addition to resist the remainder of the diagonal tension.

Let t be the safe stress in diagonal tension for a bar inclined at about 45° to the horizontal. The forces in the bars resisting diagonal tension are then tA_1, tA_2, tA_3, tA_4.

If vertical stirrups are used the force in each may be taken as $\cdot 7t \times A_s$.

From C (*Fig.* 3) set off CD equal to tA_1.
,, D ,, ,, ,, DE ,, ,, tA_2.
,, E ,, ,, ,, EF ,, ,, tA_3.
,, F ,, ,, ,, FG ,, ,, tA_4.

Beyond G set off sufficient lengths equal to $0{\cdot}7\ tA_s$ to take up the rest of the shear force represented by BC. Join CA. Through D,E,F,G,H, etc., draw lines parallel to $C.A.$, cutting AB in D_1, E_1, F_1, G_1, H_1, etc.

Then, by construction, AB is divided up into segments proportional to the forces resisted by the various bent-up bars and stirrups.

On AB describe a semi-circle.

From D_1, E_1, F_1, G_1, H_1, etc., draw perpendiculars D_1D_2, E_1E_2, F_1F_2, G_1G_2, H_1H_2, etc., to cut the circumference of the semi-circle in D_2, E_2, F_2, G_2, H_2, etc. With centre B and radii BD_2, BE_2, BF_2, BG_2, BH_2, etc., describe arcs to cut AB in D_3, E_3, F_3, G_3, H_3, etc. The bar of area A_1 should be placed midway between A and D_3, the bar of area A_2 midway between D_3 and E_3, and so on.

Proof:—Let EF (*Fig.* 2) represent in order and magnitude the force that can be resisted by a bar of area A. The bar should be placed midway between C and D and the force resisted by it is represented by the shaded portion of the diagram PQR (*Fig.* 1), viz.:—*abcd*.

$$\frac{EF}{AB} = \frac{t.A.}{\frac{1}{2}\, qb.L.}$$

$$\therefore\ \frac{a}{L} = \frac{2t.A}{qbL}.$$

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Therefore $t.A$ {the force in the bar under consideration} $= \frac{q.b.a}{2.}$.

From the right-angle triangle AF^1B :—

$$\frac{FB}{BF^1} = \frac{BF^1}{AB}$$

$$\therefore BF^1 = \sqrt{AB.FB}$$

$$= \sqrt{L.x} = BD$$

$$\text{Similarly } BC = \sqrt{L.(x+a)}$$

$$\text{Shear stress at } C = q.\frac{BC}{AB} = q\frac{\sqrt{(x+a)L}}{L} = q\frac{\sqrt{x+a}}{\sqrt{L}}.$$

$$\text{Similarly shear stress at } D = q\frac{\sqrt{x}}{\sqrt{L}}.$$

The shear force represented by the shaded portion $abcd$ = Mean stress on $CD \times b. \times CD$ [where b is the breadth of the beam].

$$\text{Hence the shear force} = \frac{q}{2\sqrt{L}}\left\{\sqrt{x+a} + \sqrt{x}\right\} b \times CD$$

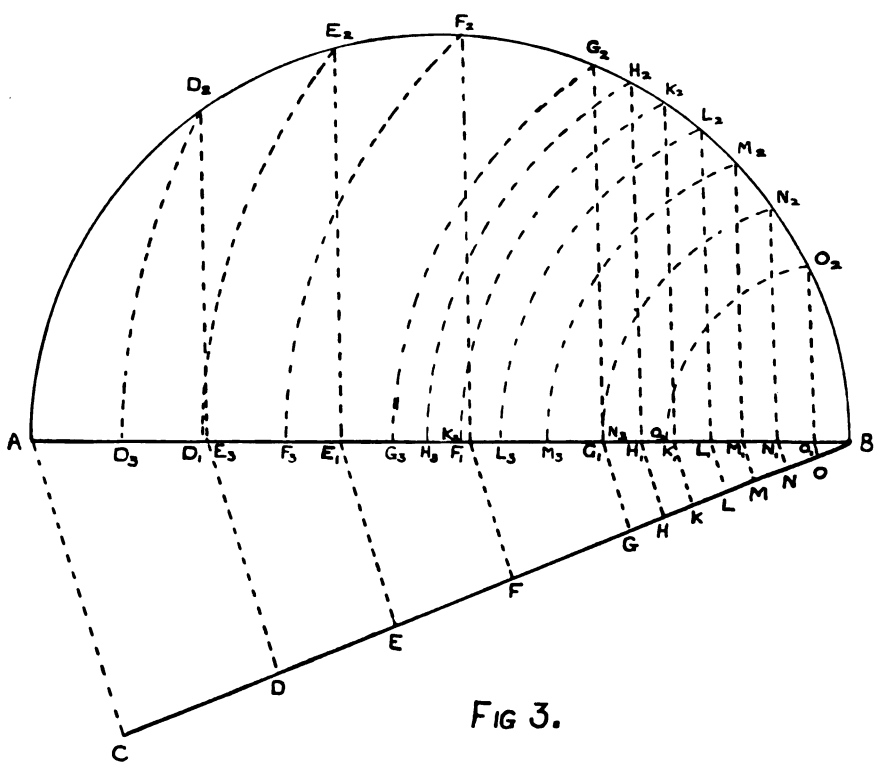

FIG 3.

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But $CD = BC - BD = \sqrt{L}\{\sqrt{x+a} - \sqrt{x}\}$.

Shear force $= \frac{q}{2\sqrt{L}}\{\sqrt{x+a} + \sqrt{x}\}b\sqrt{L}\{\sqrt{x+a} - \sqrt{x}\}$.

$$\text{Shear force} = \frac{q.ba}{2}.$$

This is the force in the bar under consideration; hence if it is placed midway between C and D it will resist all the diagonal tension represented by the area *abcd*. With inclined bars a vertical line from the middle point of CD should cut the centre point of the inclined portion of the bar.

It is usual in designing diagonal tension reinforcement to make an allowance for the resistance of the concrete. In that case the base AB (*Fig.* 3) should be set off equal to L_1, a fraction of the whole span L, given by the equation :—

$$\frac{L_1}{L} = \frac{q - \triangle c}{q}$$

where $\triangle c$ is the stress in the concrete resisting diagonal tension.

The spacing of the bent-up bars determined as above should be checked by consideration of the bending moment diagram. In nearly all cases the points at which the bars should be bent from consideration of the shear force diagram will be on the safe side of those given by the latter.

Levelling Concrete Roads.

[The above illustration shows a method of levelling concrete roads with rollers travelling on the side forms. This method is commonly used in the United States.]

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The Bridge Completed.

[*Mr. Sidney Little, A.M.Inst.C.E., Borough Surveyor.*

Reinforcement of Bow-String Beam assembled.

Reinforced Concrete Bridge at Ipswich. (*See p.* 427.)

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Stoke Bridge, Ipswich.

THERE was a bridge on the site of the present Stoke Bridge at Ipswich as early as the Norman Conquest, and reference is made to it in Domesday Book. The first definite information we have of the nature of any of the bridges at this site is the three-arch bridge which was washed away by a flood in 1818. This bridge was replaced by the one which has served up to the present, and was built to the design of the late William Cubitt, an Ipswich engineer, who it is interesting to note was also the inventor of the tread-mill. The span of the bridge was 66 ft. and its width 22½ ft. It consisted of a series of arched cast-iron Tee-ribs in two sections jointed at the crown; between these Tee-ribs cast-iron plates were placed, the ribs having both diagonal ties and cross-bracing, all in cast iron. The abutments were of a very substantial character, and more than 8 ft. thick at the springing. They were built of brickwork finished with granite skew-backs. The cost of this structure was £3,850.

The question of a new bridge to replace that built by Cubitt was considered by the Town Council for a long time previous to the war, and considerable discussion took place on the question of whether the new bridge should be "fixed" or of the "lifting" type to facilitate navigation of the upper reaches of the river. It was finally decided to have a fixed bridge, but a condition was made that the head-room should be increased by 2 ft. and that the width should be increased. These conditions added in no small measure to the difficulties of design. A design for a reinforced concrete arched bridge, with a span of 70 ft. and a width between the parapets of 42 ft., was prepared in the early part of 1914, but the war intervened and the work was postponed. After the war prices had considerably increased, and the present Borough Surveyor went into the question again to see if a more economical scheme could be devised, and the new bridge is the result.

The chief features of this bridge are:

The depth of concrete construction is only 2 ft. 3 in., thereby providing the increased headroom required without making too severe the gradient of the bridge approaches, which was governed on the town side by a neighbouring level crossing.

[*Mr. Sidney Little, A.M.Inst.C.E., Borough Surveyor*

Reinforced Concrete Bridge at Ipswich.

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The old cast-iron bridge was stripped and used for supporting the falsework of the new structure.

The old abutments were used for carrying the new work. This in itself effected a considerable saving, as will be understood when it is stated that foundation piles were 40 ft. long.

The increased river-traffic width required was obtained by having a flat soffit instead of an arched form of construction, it being considered that 60 ft. wide with this type was at least equal to 70 ft. when an arched form was used.

The following is a brief description of the new bridge: Twin piles, 16-in. square and 40 ft. long, were driven on the up-stream and down-stream side of each abutment, the centres of the piles being 5 ft. 2 in. from the edge of the abutments. Twin beams were laid over these piles and across the abutments. These beams were 9 ft. deep and reinforced top and bottom in order that should either the abutment or the piles "give" a little they would prevent cracking, and it was further considered that owing to the great depth of the beam it would be so free from deflection as to distribute a load over a considerable length of the abutment. The main bow-string beams landed on these abutment beams approximately midway between the piles and the edges of abutments.

The top boom of the main bow string is 2 ft. 3 in. by 2 ft. 3 in. with eight 1⅝-in. bars and ½-in. binding. The concrete was mixed 2 : 1 : 1; and the cover on the main bars is a minimum of 1½-in. The web-bracing is of the same composition, and in order to reduce secondary stresses as far as possible these members are made as wide and shallow as practicable. The carriageway consists of ordinary Tee-beams with 6½-in. decking made of concrete mixed 3 : 1½ : 1. The footways are cantilevered out from the main beam, the distance from the centre of the main beam to the outside of the parapet being 10 ft. The cantilevers were made *in situ*, and the flags, which were pre-cast, were fixed later.

A considerable number of conduits had to be dealt with, consisting of a 9-in. and 5-in. water main, 12-in. sewer, 5-in. compressed-air main, and four lines of pipes for electric cables and telegraph cables. These conduits were laid in their final position early, and the new bridge built over them. Daily samples of the concrete were taken and very uniform results were obtained.

Each consignment of the cement used, which was supplied by Messrs. Wm. Brown & Co. (Ipswich), Ltd., was tested. No seven-day test gave a breaking strain less than 900 lbs. per sq. in., and went as high as 940 lbs. The minimum steel used was 28·34 tons per sq. in., and varied between that and 31·5 tons. The complete cost of the bridge, property, temporary foot bridge and alterations to conduits was about £22,500.

The design and working drawings for the bridge were prepared by Mr. Sidney Little, A.M.Inst.C.E., Borough Engineer and Surveyor. The contractors were Messrs. D. G. Somerville & Co., Ltd., 35, Grosvenor-place, S.W.1, and Mr. H. Piper acted as clerk of works.

Hospital of Concrete.

THE important extension to Tilbury Seamen's Hospital, which was recently opened, is built entirely of concrete, erected on reinforced concrete rafts. The rafts are 8 ins. thick, and have a total surface area of 24,000 sq. ft. For the construction of the walls the contractors employed a system of triangular blocks which, in order to speed up the work, were made of rapid-hardening "Ferrocrete."

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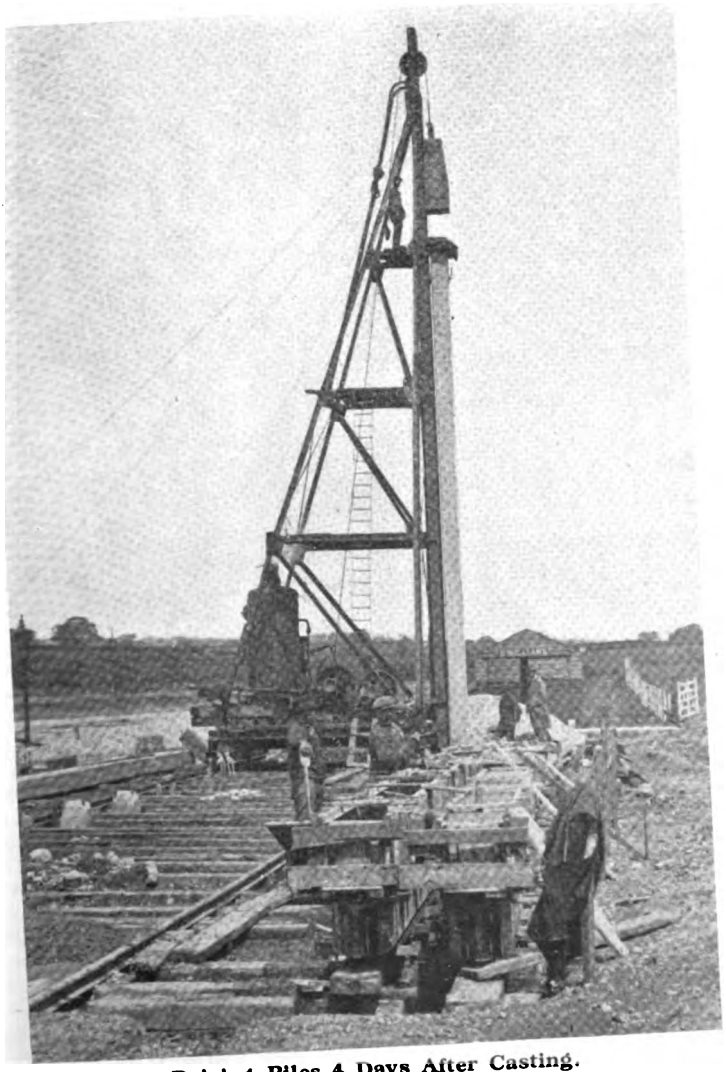
Grain Store at Newport, Isle of Wight.

THERE are several interesting features in connection with the construction of a grain store now being built at Newport, Isle of Wight, for Messrs. Thomas, Gater & Bradfield. The site is a quay on the river Medina, where the foreshore was filled in and the old quay built a generation ago under tidal conditions. It was impossible to get any foundation other than by driving piles into the blue slipper clay common to the Isle of Wight, and owing to the restricted nature of the site rapid hardening cement ("Ferrocrete") was used for the purpose.

The piles were 40 ft. long by 14 in. by 14 in., and were composed of a 1 : 1½ : 3 mixture with Langstone Harbour ballast as the aggregate. The shuttering was struck from the piles in from 12 to 14 hours after they were cast, and some of them were slung and driven in from 4 to 7 days after casting. The weight



Driving Piles 4 Days After Casting.

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of the monkey was 50 cwt., with a drop of 3 ft. at approximately 40 blows per minute. The set obtained varied from one-fifth to one-quarter inch per ten blows. Test piles were loaded with 70 tons of scrap without any movement taking place.

The grain store is to be erected on a reinforced concrete beam and raft construction superimposed on the piles.

The architects are Messrs. Stratton & Millgate, of Newport. Messrs. Frank Bevis, Ltd., of Portsmouth, are the contractors for the piling and reinforced concrete.

Book Review.

"Structural Engineering: Fundamental Properties of Materials." By S. F. Swain.
London: McGraw-Hill Publishing Co., Ltd. Pp. 200; 49 illus. Price 12s. 6d. net.

THIS volume is the second of a series by Professor Swain of Harvard University on structural engineering. It is understood that third and fourth volumes dealing respectively with the theory and design of elementary structures and with some of a more complicated character are in course of preparation. Professor Swain's works will be familiar to many who are in the habit of studying the literature of the American professional societies, and the present volume is well up to his usual standard.

The book deals with the fundamental properties of the principal materials used in structural engineering, an intimate knowledge of which should be possessed by all civil engineers. Considering the vastness of the subject, the author has, in a volume of handy size, been most successful in compressing a great quantity of information of exceeding utility. The point of view of the practical engineer has been emphasised throughout, and it is of interest to note what are considered to be the most important portions of the subject for engineers. The subjects dealt with are wood, iron—cast, wrot, and malleable—steel, alloy steels, non-ferrous metals and alloys, stone, brick and other clay products, cement and concrete, corrosion, paints and varnishes.

The volume contains such a large amount of information in condensed form that it is not possible in the space available to consider the work in detail, but the following indicates the leading features:—Under the chapter on "Wood" much useful data is given relating to the growth, felling, seasoning and cutting of timber, its defects, strength and allowable stresses, decay and preservation. Attention is drawn to the effects of defects upon strength, i.e. the influence knots, shakes and suchlike defects have in different situations on beams and columns, consideration of which is perhaps not given sufficient prominence in many cases in practice. Chapter III, on the constitution, heat treatment, and mechanical treatment of iron and steel, is worth careful study, the point the author very rightly emphasises being that "Two steels may have the same composition, but very different constitutions and physical characteristics, depending upon the heat treatment or the mechanical treatment to which they have been exposed." He therefore deals at some length with annealing and tempering and the effects of mechanical work on iron and steel, i.e. rolling, forging, hammering, and welding.

The chapters on steel deal briefly but concisely with the processes of manufacture, the effects of impurities, and the strength and ductility of steels of various compositions and constitutions; while some instructive tables are given of the physical properties of carbon, nickel and chrome steels showing the significance of specifications to secure the desired properties in steels for various purposes. In view of the increasing use of alloy steels in structural engineering, Chapter VIII will be of interest to students, but of course the use of such steels is not yet general except for very large bridges.

Chapter X deals with building stones, and tests made by various authorities for ascertaining the effect of shape and dimensions on strength and the effect of loading part of the section are discussed.

A further chapter is devoted to cement and concrete and puts in brief form the nature and properties of calcareous lime mortars and Portland cement and concrete, their manufacture, compositions, tests and strength, gauging and placing.

The concluding chapter deals with the corrosion of metals—paints and varnishes, and a few words upon water-proofing of masonry and concrete. The book can be recommended to all students of structural engineering, professors in technical colleges and to those actually engaged in the profession as an instructive work on the properties of materials.—H.F.C.

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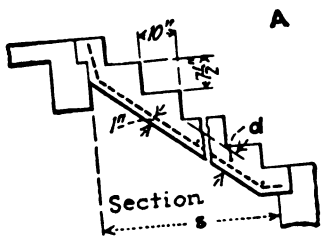
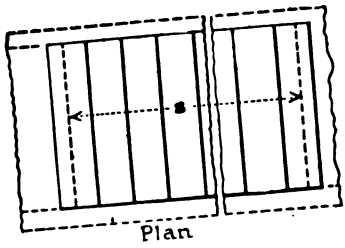
Concrete Stairs.

THE accompanying tables for the design of reinforced concrete stairs were given recently by Mr. Jacob Field in "Engineering News-Record." The average case of 10-in. treads and 7½-in. risers is used in all tables. The concrete slab is designed to carry the actual weight of the slab and steps at 144 lb. per cubic foot and 100 lb. per sq. foot, horizontal projection, as live-load. The slabs are considered as simple spans; landings are part of the span. In cases where a landing is common to two runs of stairs, a load equal to half the actual dead-load of the landing plus 50 lbs. per sq. ft. live-load over the length of the landing is carried by each span. The effective depth of the stair slab is measured normal to the slope of the stair run, allowing 1 in. covering for the reinforcement; the depth of the landing is measured vertically. The effective depth of stairs and landing is the same in all cases. The unit maximum fibre stresses are 16,000 lbs. per sq. in. for the steel and 650 lbs. per sq. in. compression for the concrete. The usual formulæ resulting from the assumptions of no tension in the concrete and a straight line stress variation across normal sections of the slabs are employed. The following cases are covered:

Table A—Straight stair run, from 4 to 23 treads. The end risers are assumed 5 in. from the inner surface of the supporting beams. This distance is usually 4 or 6 in., depending upon the designer, and 5 in. is taken as an average value. The spans are measured horizontally and





L.L. 100 lb. per sq. ft.
f_c = 650 lb. per sq. in.
f_s = 16,000 lb. per sq. in.

s = Span, d = thickness of slab
A_s = Area of steel per foot width of stairs
C = Volume of concrete in cu. ft. per ft. width of stairs
S = Weight of steel (incl. 3/8" dist. steel, 20" on center) per foot width

Number Treads	Span Ft.	Span In.	d In.	A_s Sq. In.	C Cu. Ft.	S Lb.	Spacing of Bars, In. ⅜ In. φ	½ In. φ	½ In. □	⅝ In. φ	Span, In.
4	4	2	4½	0.11	3.7	2.9	10	..	..	..	50
5	5	0	4½	0.16	3.9	4.0	8	..	..	..	60
6	5	10	4½	0.22	4.6	5.9	6	11	..	..	70
7	6	8	4½	0.29	5.2	8.1	4½	8	11	..	80
8	7	6	4½	0.35	6.1	10.8	3½	7	8½	10½	90
9	8	4	5¼	0.39	7.2	13.2	3¼	6	7½	9½	100
10	9	2	5¾	0.44	8.4	15.9	3	5½	7	8½	110
11	10	0	6¼	0.49	9.6	19.1	2¾	4¾	6¼	7½	120
12	10	10	6¾	0.54	11.0	22.8	2½	4¼	5½	6¾	130
13	11	8	7½	0.58	12.7	26.1	2¼	4	5¼	6¼	140
14	12	6	8	0.64	14.3	30.2	2	3¾	4¾	5¾	150
15	13	4	8½	0.70	16.0	35.2	..	3¼	4¼	5¼	160
16	14	2	9¼	0.74	18.1	40.1	..	3	4	5	170
17	15	0	9¾	0.91	19.9	45.2	..	2¾	3¾	4½	180
18	15	10	10½	0.86	22.2	50.3	..	2¾	3½	4¼	190
19	16	8	11	0.91	24.3	54.6	..	2½	3¼	4	200
20	17	6	11¾	0.98	26.8	63.7	..	2¼	3	3¾	210
21	18	4	12¼	1.05	29.1	72.7	..	2	2¾	3½	220
22	19	2	13	1.12	32.0	80.9	..	2	2½	3¼	230
23	20	0	13¾	1.18	34.8	87.3	..	..	2½	3	240

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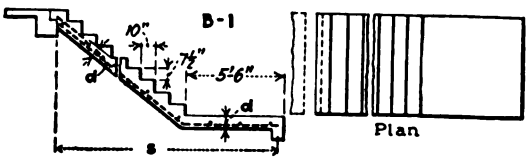
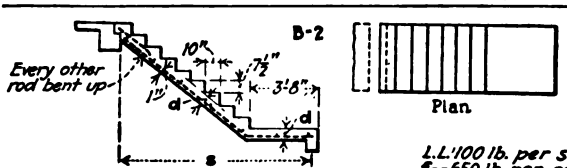


Section

Number Treads	Span Ft.	Span In	d In.	A_s Sq.In.	C Cu.Ft	S Lb	Spacing of Bars, In. ½ In φ	½ In φ	½ In. □	½ In. φ	Span, In.
0	5	6	4	0.16	1.8	3.0	8¼	..	.	..	66
1	6	4	4	0.22	2.4	5.0	6	11			76
2	7	2	4	0.30	3.1	7.8	4½	8	10	..	86
3	8	0	4½	0.34	4.0	9.9	4	7	9	11	96
4	8	10	5	0.39	5.1	12.5	3½	6	7½	9½	106
5	9	8	5½	0.45	6.2	15.8	3	5½	6½	8	116
6	10	6	6¼	0.47	7.7	18.1	2¾	5	6	7½	126
7	11	4	6¾	0.53	9.0	21.8	2½	4½	5½	7	136
8	12	2	7¼	0.58	10.5	25.9	2¼	4	5	6	146
9	13	0	7¾	0.64	12.0	30.1	2	3¾	4¾	5¾	156
10	13	10	8¼	0.69	13.6	34.4	..	3½	4½	5¼	166
11	14	8	8¾	0.75	15.3	39.4	..	3¼	4	5	176
12	15	6	9½	0.80	17.4	44.3	..	3	3¾	4½	186
13	16	4	10	0.85	19.3	49.6	..	2¾	3½	4¼	196
14	17	2	10½	0.91	21.6	56.0	..	2½	3¼	4	206
15	18	0	11¼	0.97	23.8	62.3	..	2¼	3	3¾	216
16	18	10	12	1.04	26.4	69.5	..	2¼	2¾	3½	226
17	19	8	13	1.14	29.6	79.7	..	2	2½	3¼	236



Section

L.L.=100 lb. per sq. ft
f_c=650 lb. per sq. in.
f_s=16,000 lb. per sq. in.

s=Span; d=thickness of slab
A_s=Area of steel per foot width of stairs
C=Volume of concrete in cu.ft per foot width of stairs
S=Weight of steel (incl ⅜" dist steel, 20" on center) per foot width

Number Treads	Span Ft.	Span In.	d In.	A_s Sq.In.	C Cu.Ft.	S Lb.	Spacing of Rods, In. ½ In φ	½ In. φ	½ In. □	½ In. φ	Span, In.
0	3	8	4	0.13	1.2	2.8	10	..		..	44
1	4	6	4	0.14	1.8	2.9	9½		..	..	54
2	5	4	4	0.17	2.4	4.1	8		..	..	64
3	6	2	4	0.23	3.1	5.9	5¾	10	..	..	74
4	7	0	4½	0.31	3.8	8.8	4½	7½	9½	..	84
5	7	10	4½	0.37	4.6	11.3	3¾	6¼	8¼	10	94
6	8	8	5	0.43	5.7	14.6	3½	5¾	7	8¾	104
7	9	6	5½	0.45	6.9	16.5	3	5¼	6½	8¼	114
8	10	4	6¼	0.49	8.3	19.5	2¾	4¾	6	7½	124
9	11	2	6¾	0.53	9.6	22.6	2½	4½	5½	7	134
10	12	0	7½	0.58	11.4	26.5	2¼	4	5¼	6¼	144
11	12	10	8	0.64	12.9	30.6	2	3½	4½	5¾	154
12	13	8	8¾	0.69	14.9	35.0	..	3¼	4¼	5½	164
13	14	6	9¼	0.75	16.7	39.9	..	3¼	4	5	174
14	15	4	10	0.79	18.9	44.8	..	3	3¾	4¾	184
15	16	2	10¾	0.83	21.2	49.1	..	2¾	3½	4½	194
16	17	0	11¼	0.89	22.9	55.0	..	2½	3¼	4¼	204
17	17	10	12	0.91	25.8	58.9	..	2½	3¼	4	214

Designs of Reinforced Concrete Stairs.

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vary from 4 ft. 2 in. to 20 ft. Each step weighs 37½ lbs. per linear foot, which is considered as an extra dead-load uniformly distributed over the span, 3·75 lbs. per in. or 45 lbs. per sq. ft. The dead-load of the main slab is 1·25 *b* lbs. per sq. ft., horizontal projection, *b* being the normal thickness of the slab in inches.

A is the area of tension steel required per foot width of stairs, in square inches.

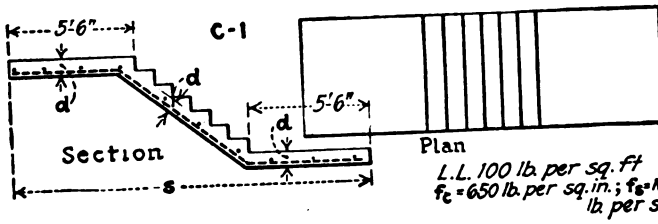
C is the volume of concrete in cubic feet per foot width of stairs.

S is the weight of steel required per foot width of stairs, in pounds. This includes the weight of the tension steel and also a ⅜-in. round rod under every second step to act as distributing steel.

Table B 1—The case of a single run of steps, varying from 1 to 17 treads, with a 66-in. landing at one end. The notation is the same as in Table A; the live-load is 100 lbs. per sq. ft. over the entire span.

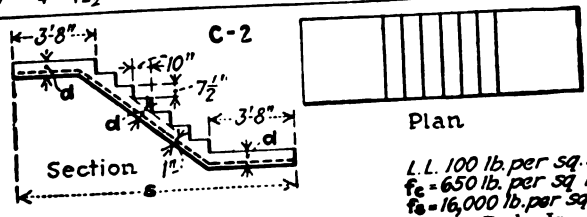
Table B 2—The same as B 1, except that the landing is 44 in. long.

Table C 1—The case of a single run of



L.L. 100 lb. per sq. ft
f_c = 650 lb. per sq. in.; f_s = 16,000 lb. per sq. in.

Number Treads	Span Ft.	Span In.	d In.	A_s Sq.In.	C Cu.Ft.	S Lb.	Spacing of Rods, In. ⅜ In. φ	½ In. φ	½ In. □	⅝ In. φ	Span, In.
0	11	0	6	0.45	5.5	16.8	3	5¼	6½	8	132
1	11	10	6½	0.51	6.7	20.8	2½	4½	6	7	142
2	12	8	7	0.57	8.1	25.0	2¼	4	5¼	6½	152
3	13	6	7¾	0.63	9.8	29.6	2	3¾	4¾	5¾	162
4	14	4	8½	0.67	11.6	33.5	2	3½	4½	5½	172
5	15	2	9	0.74	13.2	39.1	..	3¼	4	5	182
6	16	0	9¾	0.79	15.4	44.1	..	3	3¾	4½	192
7	16	10	10¼	0.88	17.2	51.8	..	2¾	3½	4¼	202
8	17	8	11	0.92	19.4	59 8	..	2½	3¼	4	212
9	18	6	11¾	0.99	21.9	64.0	..	2¼	3	3¾	222
10	19	4	12½	1.04	24.2	70 4	..	2¼	2¾	3½	232



L.L. 100 lb. per sq. ft.
f_c = 650 lb. per sq. in.;
f_s = 16,000 lb. per sq. in.

Number Treads	Span Ft.	Span In.	d In.	A_s Sq.In.	C Cu.Ft.	S Lb.	Spacing of Rods, In. ⅜ In. φ	½ In. φ	½ In. □	⅝ In. φ	Span, In.
0	7	4	4¼	0.26	2.6	7.3	5	9	..	..	88
1	8	2	4¾	0.32	3.5	9.9	4¼	7½	9½	..	98
2	9	0	5¼	0.37	4.6	12.4	3½	6½	8	10	108
3	9	10	5¾	0.42	5.7	15.4	3¼	5½	7½	8¾	118
4	10	8	6¼	0.48	6.9	18.7	2¾	5	6¾	7¾	128
5	11	6	6¾	0.54	8.2	22.8	2½	4½	5½	7	138
6	12	4	7¼	0.59	9.5	26.7	2¼	4	5	6¼	148
7	13	2	8	0.64	11.3	30.7	2	4	4¾	5¾	158
8	14	0	8½	0.70	12.8	35.4	..	3½	4½	5¼	168
9	14	10	9	0.76	14.5	41.0	..	3	4	4¾	178
10	15	8	9¾	0.81	16.6	45.6	..	3	3¾	4½	188
11	16	6	10¼	0.87	18.3	52.0	..	2¾	3½	4¼	198
12	17	4	11	0.94	20.7	58.1	..	2½	3¼	4	208
13	18	2	11¾	1.00	23.2	65.0	..	2¼	3	3¾	218
14	19	0	12½	1.06	25.8	71.6	..	2¼	2¾	3½	228

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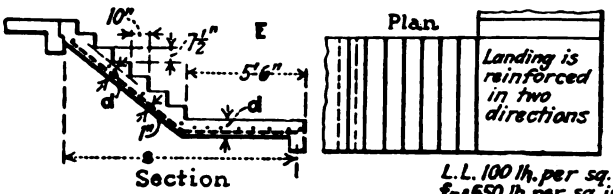


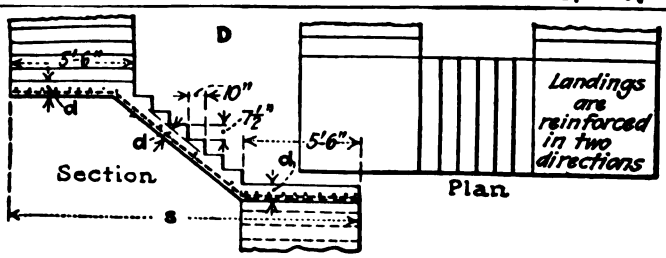
L.L. 100 lb. per sq. ft.
f_c = 650 lb. per sq. in.
f_s = 6,000 lb. per sq. in.

s = Span; d = thickness of slab

A_s = Area of steel per foot width of stairs

Include 50% of landing { C = Volume of concrete in cu. ft. per foot width of stairs
S = Weight of steel (incl. 3/8" dist. steel in stair slab, 20% lc.) per ft. width

Number Treads	Span Ft.	Span In.	d In.	A_s Sq. In.	C Cu. Ft.	S Lb.	Spacing of Rods, In. — ⅜ In. φ	½ In. φ	½ In. □	⅝ In. φ	Span, In.
1	6	4	4	0.13	1.5	2.9	10¼	..	..	..	76
2	7	2	4	0.16	2.1	4.3	8¼	..	..	..	86
3	8	0	4	0.23	2.7	6.8	5¾	10	..	..	96
4	8	10	4	0.32	3.4	10.5	4¼	7½	9½	..	106
5	9	8	4½	0.39	4.3	13.9	3¼	6	7¾	9½	116
6	10	6	5	0.45	5.3	17.2	3	5¼	6¾	8	126
7	11	4	5¾	0.48	6.6	19.8	2¾	5	6¼	7½	136
8	12	2	6½	0.53	8.1	23.5	2½	4½	5¾	7	146
9	13	0	7¼	0.58	9.7	27.4	2¼	4	5¼	6½	156
10	13	10	8	0.63	11.4	31.6	2	3¾	4¾	5¾	166
11	14	8	8¾	0.69	13.2	36.5	..	3½	4½	5¼	176
12	15	6	9½	0.74	15.2	41.3	..	3¼	4	5	186
13	16	4	10	0.81	17.0	47.5	..	3	3¾	4½	196
14	17	2	10¾	0.89	19.2	54.6	..	2½	3½	4	206
15	18	0	11¼	0.96	21.2	61.5	..	2¼	3	3¾	216
16	18	10	12	1.02	23.6	68.4	..	2¼	3	3½	226
17	19	8	13	1.13	26.6	78.8	..	2	2¾	3¼	236



Number Treads	Span Ft.	Span In.	d In.	A_s Sq. In.	C Cu. Ft.	S Lb.	Spacing of Rods, In. — ⅜ In. φ	½ In. φ	½ In. □	⅝ In. φ	Span, In.
1	11	10	6	0.31	6.0	12.9	4	7½	9½		142
2	12	8	6	0.42	6.6	18.3	3	5¾	7	8¾	152
3	13	6	6½	0.49	7.7	23.3	2½	4¾	6	7¼	162
4	14	4	7	0.56	8.9	28.1	2¼	4	5¼	6½	172
5	15	2	7¾	0.62	10.6	33.0	..	3½	4¾	5¾	183
6	16	0	8½	0.71	12.4	39.5	..	3¼	4¼	5¼	192
7	16	10	9¼	0.74	14.3	43.9	..	3	4	4¾	202
8	17	8	10	0.81	16.3	50.1	..	2¾	3½	4½	212
9	18	6	10¾	0.88	18.9	57.1	..	2½	3¼	4	222
10	19	4	11½	0 94	20.8	63.7	..	2¼	3	3¾	232

Designs for Reinforced Concrete Stairs.

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stairs, varying from 1 to 10 treads, with a 66-in. landing at each end.

Table C 2—The same case as C 1, except that the landings are 44 in. long.

Table D—The case is similar to that in Table C, but assumes that each landing is common to two-stair runs, hence only half of the load acting upon the full landing length is considered. By this method, and considering full spans, duplication of load is avoided. No distributing steel is assumed in the landings, which are really two-way reinforced slabs. *C* denotes the volume of concrete per foot width of stairs, including half the volume of each landing.

Table E—This case is similar to that in Table D, for one landing common to two runs of stairs.

AN "UMBRELLA" LOCOMOTIVE WATERING TANK.

By S. A. BUNTING, M.A., M.Inst.C.E.

When a tank for watering locomotives has to be erected between tracks whose distance apart only slightly exceeds running dimensions, a ferro concrete "Y" tank offers a cheap solution.

This design, as built, is for a 1200-gallon tank, but no difficulty would be experienced in quadrupling the capacity without any change in principle. It is illustrated on page 436.

Four rails are bent and drilled as shown, welded bands threaded over them in the pedestal portions, and the foundation concrete and the pedestals cast. Through the top fishbolt holes a bar bent "U" shape is passed; the arms of the "U" are then bent in to lap each other 2 or 3 ft. and are clamped tightly together by broken fishplates and fishbolts. For a tank of this size using 41¼-lb. rails one more "U" bar at least must be similarly threaded through the rails and clamped, about two-thirds the depth of the tank down. Larger tanks might do well with positive support of this nature at yet a third point. For convenience two tie-rods through the second fishbolt holes hold the pairs of rails at the exact distance required.

For the rest, reinforcing rectangles slightly rounded at the corners and depressed towards the outer surface in the centres of sides are laid in the "Ys" and wired if necessary. These are calculated as for floor slabs.

The outer forms are clamped on to the rails with bolts and broken fishplates, the nuts inside; pieces of stone the cover-thickness required are inserted as packing between the forms and the rails.

The forms for the ends are bolted through to the corresponding outer forms. The necessary pipes and valves are set in position and the concreting commenced. As the bottom and the lower portions of the sides get filled boards previously cut to length are laid on the concrete as work proceeds, allowing men to work inside the tank. A few sand-bags, or, if desired, a couple of light triangular frames, will hold these boards in position and allow a certain amount of tamping behind them. Better concrete results this way than if full inner forms are fixed in advance.

This design worked out at about one-third the cost of the cheapest steel structure of the kind. Two tanks have so far been cast with the same forms.

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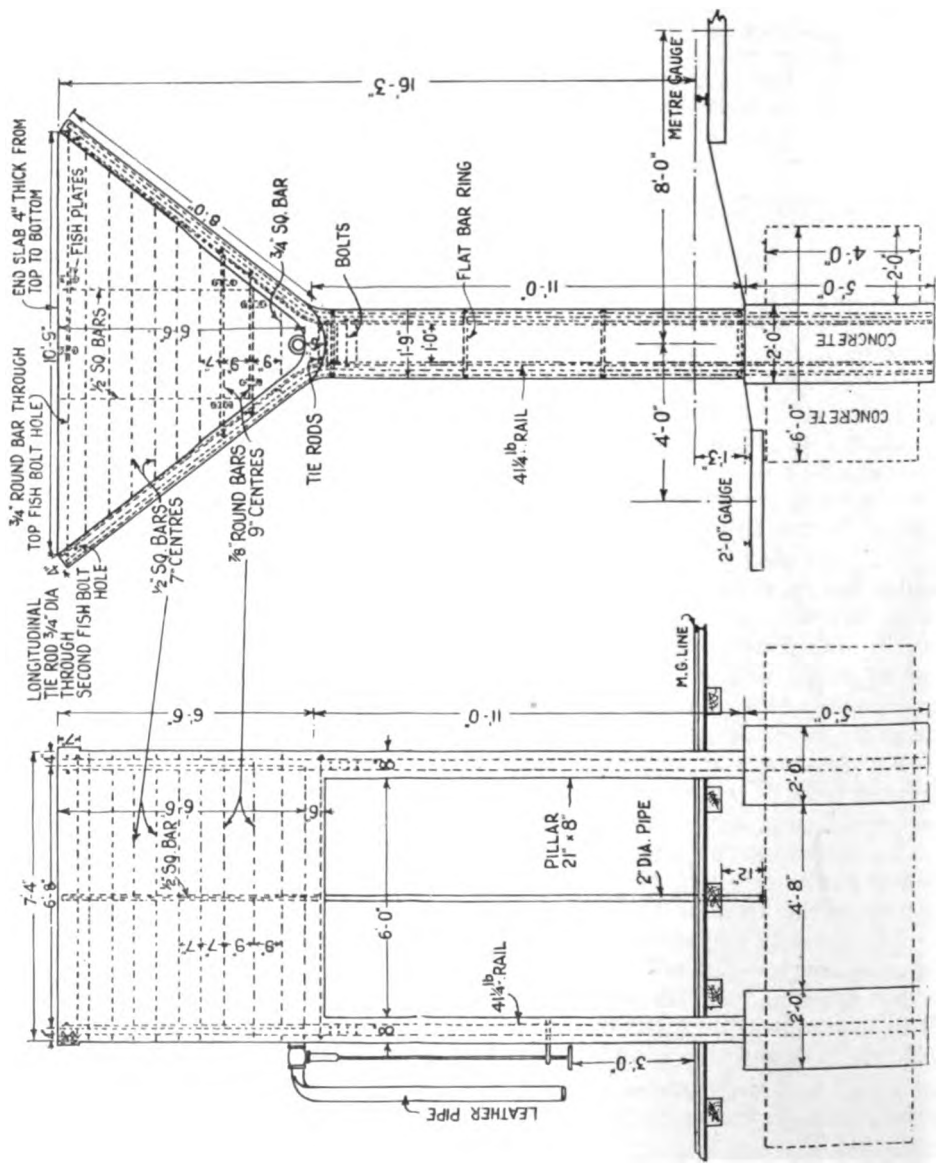

An "Umbrella" Locomotive Watering Tank. (See p. 435.)

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The Value of Concrete in Reinforced Concrete.

In the course of a paper communicated to the Institution of Civil Engineers, on "The Value of Concrete in Reinforced Concrete Structures," Mr. W. A. Smith, A.M.Inst.C.E., first refers to the results of tests carried out from time to time on concrete materials as used in connection with various reinforced concrete construction works undertaken by the Bristol Docks Committee at Avonmouth. Information has been obtained from the tests as to the crushing strengths and comparative costs of different qualities or mixtures of concrete; and these data are utilised in investigating whether or not it would be more economical to use a richer mixture of concrete in reinforced concrete structures, at the same time taking advantage of the higher compressive stresses which the tests have shown may safely be used in the calculations.

In order to ascertain to what extent the weight of cement was ultimately influenced by the presence of water, an experiment was carried out with a 6-in. cube made entirely of neat cement and water. Three lbs. of water were used in mixing with 11¼ lbs. of dry cement, and after 40 days 2½ lbs. of water were still retained in the cube.

A number of crushing tests were carried out with 4-in. cubes of 4 to 1, 5 to 1, and 6 to 1 mixtures. In all more than 130 cubes were crushed, the results being tabulated. In some of the tests the cement was gauged by volume, whilst in others it was weighed, and the comparative crushing results show the advantages in measuring the cement by weight. In practically all cases the concrete was machine mixed.

A series of tests was carried out to arrive at the relative costs of the different mixtures of concrete. Five tests were made with the 4 to 1 mixture, three with the 5 to 1, and two with the 6 to 1 concrete. The ratio of coarse to fine aggregate was slightly varied in the different tests, and two sands were used as the fine aggregate, with the exception of one of the 4 to 1 mixtures, when three sands formed the fine aggregate.

The data arrived at from the tests are utilised to investigate the relative costs of various typical members of a reinforced concrete structure using different qualities of concrete. The following working stresses were adopted for the various mixtures:—

Mixtures.	Compressive Stress.
4 to 1	800 lbs. per sq. in.
5 to 1	700 lbs. per sq. in.
6 to 1	600 lbs. per sq. in.

Whilst the author is of the opinion that higher stresses might safely be used for the richer mixtures of concrete, as shown by the results of the crushing tests, the above figures, which are more or less in keeping with those recommended by the R.I.B.A., have been adopted in the calculations for the typical examples of the reinforced concrete members referred to. The tensile stress in the steel is taken at 16,000 lb. per sq. in.

Typical designs of both rectangular and tee beams, made with different qualities of concrete, are compared, the safe compressive stress in the concrete being in keeping with the richness of the mixtures. The relative costs per lineal ft. of each of the beams are given, and the results show that the richer the mixture the cheaper the design, due to the fact that a higher permissible compressive stress is assumed with the richer mixtures.

The author considers that in reinforced concrete beams where top and bottom reinforcement is necessary the compression steel is not as a rule used to the full advantage. The general practice in designing such beams is to assume that the stress-strain curve is a straight line, and the intensity of stress on the compression steel is usually taken as being 15 times the intensity of stress on the surrounding concrete. Therefore the stress on the concrete at the point where the steel is placed, being directly proportional to its distance from the neutral axis, is materially reduced, more particularly in shallow beams; and consequently a corresponding reduction in the permissible stress on the compressive reinforcement has also to be made. It is, however, sometimes recognised that the stress-strain curve is not a straight line, but approaches a parabolic curve with the vertex at the compression edge of the beam. This, it is claimed, is more accurate and such has proved to be the case from results of experiments on rectangular beams of steel and iron. If this is taken into account in reinforced

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concrete beams, the average stress over the compression area of the concrete and the compressive stress in the steel would be greatly increased. Two beams of equal external dimensions, numbered respectively Nos. 13 and 14, were designed, No. 13 by the usual practice on the assumption that the stress-strain curve is a straight line, and No. 14 on the assumption that the stress-strain curve is parabolic. The results show that by providing approximately 30 per cent. more tensile reinforcement in beam No. 14, with a corresponding increase in cost of 4 per cent., the beam will theoretically resist a superimposed load 30 per cent. in excess of the load that beam No. 13 will carry.

In the design of reinforced concrete columns the author recommends that the full cross-sectional area of the columns should be taken into account in the calculations, as considerable saving in the cost of the work would be effected by adopting this method. Some typical designs of columns are considered, and, as is the case with beams, the columns made with the richer mixtures are more economical, but with columns the saving is much more pronounced.

The main reinforcing rods of a column are assumed to be stressed to fifteen times the intensity of stress of the surrounding concrete, and this increased stress is imparted to the rods by the adhesion between the concrete and steel. The load on a column is, practically speaking, of the same intensity throughout its length, and, considering the uppermost portion of the column and the comparatively short lengths of rods which project up into the superstructure, it is difficult to imagine how this increased stress is transmitted to the steel at this point. The conditions are somewhat different on the compression side of a beam, in which case the stress comes on gradually, and the compression steel is usually of sufficient length to ensure proper adhesion. It is essential, however, to provide a certain amount of main reinforcing steel in a column to resist any lateral stresses that may be induced, and also to act as anchorages for the bindings, but if the main rods were reduced to a minimum and more consideration were given in the design of a column with a view to providing more hoopings or lateral bindings, it is considered that better results would be achieved and at the same time a saving would be effected on the cost of the work. Higher stresses could safely be taken on the concrete, more particularly on the area enclosed by the bindings.

Concrete Houses Poured in Twelve Hours.

A MONOLITHIC concrete house has been built for the Winchester Corporation and will be used to accommodate one of the Council's staff. The concrete for the entire house, with the exception of the foundation and bottom flooring, was poured in at one operation. After the foundation and sub-floor had set the inner moulds were erected. These moulds are of such dimensions as will allow them to be handled by two men. At this point the bulk of the fittings are placed in position—window and door frames, conduits for electric wiring, gas pipes, ventilators and chimney flues. Grooves for water and drain pipes are also formed, and reinforcing rods inserted where necessary. When this has been completed the outer moulds are erected. The process of setting up the moulds for a small house takes about 60 hours, and the actual pouring from 10 to 15 hours. By the use of rapid hardening cement the shuttering was removed within three days, after which the plastering and stucco were applied immediately. The floors are then placed and the roof erected over the concrete ceiling, which is poured in the same operation as the walls. The window casements, doors, and all interior fittings are then added.

The constructors of the house, Monolithic Concrete Houses, Ltd., estimate that the average price of these houses should work out at 20 per cent. below that prevailing for brick or steel houses.

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CALCIUM CHLORIDE IN CONCRETE.

THE Structural Materials Research Laboratory, Lewis Institute, Chicago, has issued a reprint of a paper by Professor Duff Abrams, read before the American Society for Testing Materials, dealing with the subject of calcium chloride as an admixture in concrete.

Strength tests of concrete and mortar were made at ages of 2 days to 3 years using admixtures of calcium chloride from two different manufacturers and three proprietary compounds of calcium-chloride base. The tests covered a wide range of mixtures, consistencies, and curing conditions, and included studies with four different brands of cement. In one series of tests magnesium chloride was included. For purpose of comparison parallel tests were made on concrete and mortar without admixtures. A few tests were made using calcium chloride in concrete cured at low temperature.

Tests were made on about 7500 compression specimens in three separate investigations, as follows:

Series 156.—Compression tests of concrete and mortar at ages of 2 days to 3 years using two different brands of calcium chloride, standard and low chlorine "Cal," "Vitriflux," and magnesium chloride in varying amounts of admixtures. Tests on one brand of calcium chloride were extended to include a wide range of mixes, consistencies, curing conditions, and four different Portland cements.

Series 158.—Compression tests of 1 : 5·2 concrete at ages of 7 days to 2½ years using calcium chloride as admixture for a wide range of periods of curing in moist room followed by air curing. The influence of calcium chloride on strength of concrete made from five different cements was also studied.

Series 188.—Compression tests of 1 : 5 concrete cured at low temperatures at ages 1 to 28 days. In certain tests calcium chloride was used as admixture.

Concrete and mortar tests were made in accordance with standardised methods. In general, each set included 5 to 10 concrete specimens or 5 mortar specimens made on different days. The strength of concrete containing admixtures was compared with the strength of similar concrete without admixtures for the same mix, age, curing condition, etc.

The principal conclusions from these tests are:

(1) Calcium chloride and the admixtures containing calcium chloride, when used within certain limiting percentages, gave increased strength to concrete.

(2) Magnesium chloride in flake form (46 per cent. magnesium chloride, 54 per cent. water) when used in concrete caused a reduction in strength except for small percentages, which showed a slight increase for 2 and 7-day tests.

(3) The effect of the calcium-chloride admixtures in concrete varied with the type and quantity of admixture. When the tests are compared on the basis of chlorine content the different admixtures give similar results but not the same numerical values.

(4) Calcium-chloride admixtures of the types used increased the strength of concrete approximately in proportion to the amount used up to a certain "optimum" percentage; the strength increase became less for further additions. Beyond a certain critical percentage further additions produced a reduction in strength as compared with similar concrete without admixtures.

(5) Optimum and critical percentages for the calcium chloride admixtures used

TABLE 1.

Admixture.	Per Cent. of Admixture by Weight of Cement.		Per Cent of Chlorine by Weight of Cement.	
	Optimum.	Critical.	Optimum.	Critical.
Commercial Calcium Chloride A	2 to 4	6 to 8	1 to 2	3 to 4
Commercial Calcium Chloride B	2 to 4	6 to 8	1 to 2	3 to 4
Standard Cal.	7 to 10	15 to 20	1 to 2	3 to 4
Low-Chlorine Cal.	about 10	15 to 20	1 to 2	2 to 3
Vitriflux	6 to 8	10 to 15	1 to 2	2 to 3

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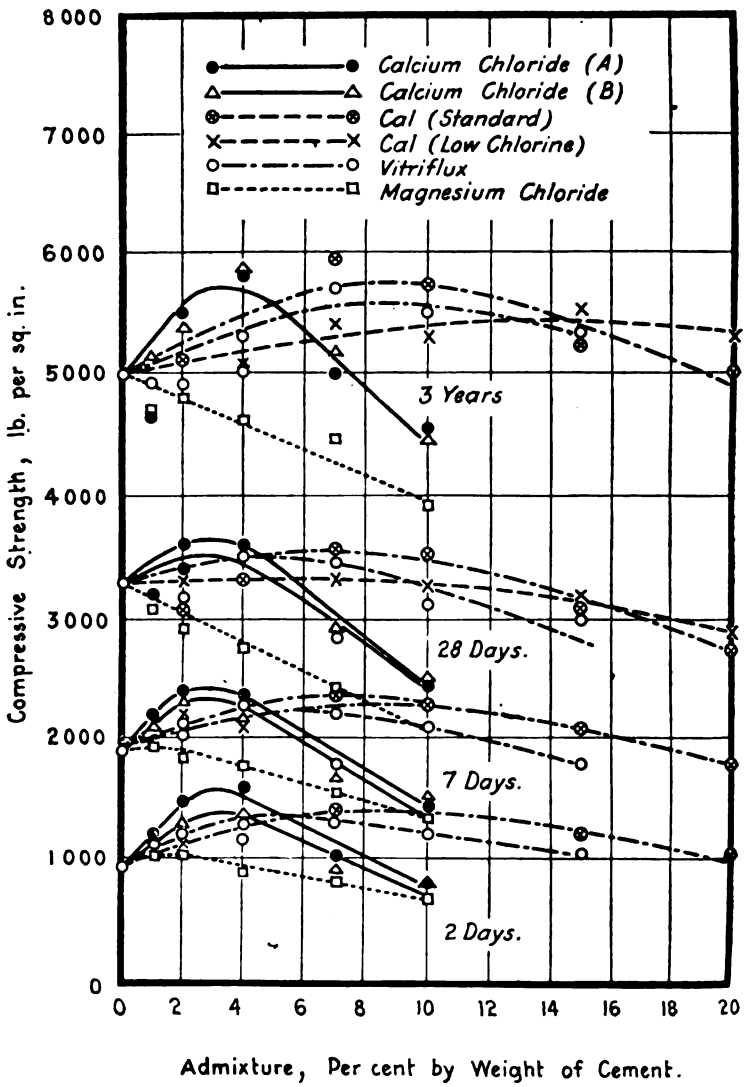

FIG. 1.—EFFECT OF CALCIUM CHLORIDE ON THE STRENGTH OF CONCRETE.

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in these tests for 1 : 5·2 concrete of normal consistency and cured in moist room, are given in Table 1.

(6) The relative effectiveness of the different admixtures when compared on the basis of strength-ratio (the ratio of the strength of concrete containing admixture to that of similar concrete without admixture, expressed as a percentage) for the optimum percentages, is shown by the strength-ratios in Table 2, which are based on tests of 1 : 5·2 concrete of normal consistency cured in the moist room :

(9) The increase in strength due to the addition of 2 per cent. of commercial calcium chloride was practically negligible for a 1 : 7 mix; it increased with the cement content up to about a 1 : 4 mix; for mixes richer than 1 : 4 the strength increase remained about the same. The strength increase for the different mixes and the corresponding strength-ratios for ages of 2 days to 3 years are given in Table 3.

(10) The tests confirm the relation between water-cement ratio and compres-

TABLE 2.

Admixture.	Strength-Ratio for Optimum Percentage.					
	2 Days.	7 Days.	28 Days.	3 Months.	1 Year.	3 Years.
Commercial Calcium Chloride A .	170	125	110	112	117	118
Commercial Calcium Chloride B .	147	124	109	109	119	116
Standard Cal.	151	123	107	107	121	119
Low-Chlorine Cal.	143	118	101	104	113	111
Vitriflux	138	115	106	108	109	114

(7) The addition of calcium chloride admixtures in amounts less than the critical percentage produced an increase in concrete strength that was practically constant at all ages. This constant strength increase, when expressed as a percentage of the strength of concrete without admixture, showed a diminishing ratio as the concrete gained in strength with age.

(8) The addition of 2 per cent. of calcium chloride to 1 : 9·2 concrete of the consistency ordinarily used in construction work (1·10 to 1·25) produced an increase in compressive strength of 100 to 200 lbs. per sq. in. at ages of 2 days to 3 years; for 1·50 consistency there was little or no increase; for consistency 0·90 the increase was about 550 lbs. per sq. in.

sive strength pointed out in other publications of this Laboratory. The water-ratio-strength curves for 2 per cent. of commercial calcium chloride were similar in form to those for plain concrete but divergent at the lower water-ratios, showing the greater effect of the calcium chloride on the richer mixes and drier concretes.

(11) The effect of commercial calcium chloride on the strength of concrete cured under different moisture conditions at normal temperatures was slightly greater in concrete cured in the air of the laboratory than in concrete cured in water or in the moist room.

(12) For concrete cured outdoors at temperatures near freezing, the use of 3 per cent. of calcium chloride produced

TABLE 3.

Mix	Strength Increase, lbs. per sq. in. (same for all ages).	Strength-Ratio per cent.				
		2 Days.	7 Days.	28 Days.	3 Months.	3 Years.
1 : 7 .	Negligible (±100)	115	107	85	88	95
1 : 52 .	400	138	125	107	110	110
1 : 4 .	700	148	130	119	117	115
1 : 3 .	1200	150	130	122	119	115
1 : 2 .	1300	149	130	122	120	105

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about the same strength increase as for other curing conditions. This strength increase appears as a much higher percentage gain when compared to the low strengths attained for outdoor curing. Too much dependence should not be placed on the use of calcium chloride for preventing freezing of concrete in cold weather; common salt should not be used for this purpose.

(13) The cements used in the tests were in general similarly affected by addition of calcium chloride.

(14) The effect of calcium chloride in mortar was essentially the same as in concrete.

(15) The time of setting of the cement used was shortened by each of the admixtures. Flash set was produced by most of the admixtures when used in percentages somewhat in excess of those found to give greatest strength increases in concrete.

(16) The important conclusion from these tests may be summarised as follows:

In the use of calcium chloride no advantage was gained for percentages of the commercial product greater than 2 or 3 per cent. of the weight of cement (chlorine content 1 to 1½ per cent.). This amount when used in mixes of about 1 : 5 and in consistencies suitable for building construction showed an increase in compressive strength of from 100 to 200 lbs. per sq. in., which increase was practically constant at ages of 2 days to 3 years. For richer mixes and drier consistencies the strength increase was greater and for leaner mixes and wetter concretes it was less.

Fig. 1 shows the effect of different types of admixture on the strength of concrete. The tests were on 6 in. by 12 in. cylinders and in a moist room, the mix being 1 : 5·2 by volume of the relative consistency 1·00.

Concrete Houses at Shepton Mallet.

[These cottages, part of the housing scheme of the Shepton Mallet Urban District Council, are built of breeze blocks (2 leaves of 3 in. block and a 2 in. cavity) and roughcast outside. The blocks were made by the Bristol Concrete Co., Ltd., on "Manelite" hydraulic machines.]

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Modern Methods of Construction.

By Oscar Faber, O.B.E., D.Sc., M.Inst.C.E.

THE following is the conclusion of the paper entitled "Modern Methods of Construction" recently read by Dr. Oscar Faber before the Royal Institute of British Architects :—

Perhaps one of the most interesting applications of rafts of this kind is to the large and important buildings being erected in Shanghai. The soil consists of a sandy clay brought down by the Yangtze River quite recently, geologically speaking, and still contains so much water as to be properly described as soft mud. The top 6 ft. has dried out a little more than below. Bore-holes have been sunk to a depth of over 400 ft., but instead of coming across more solid strata the material remains soft mud for the whole of this depth. The amazing thing is that on this soil magnificent large modern buildings, equal in size and magnificence to anything we are erecting in England, are being built. The construction in every case takes the form of a reinforced concrete raft spreading the load over the whole area of the site and somewhat beyond it. This raft takes a certain proportion of the weight of the building, and the rest is carried by piles driven into the ground before the raft is built. These piles do not rest on anything more solid, but add considerably to the resistance in virtue of the additional skin friction between them and the mud; or, looking at it from another point of view, they transfer part of the pressure from near the surface where the carrying capacity of the soil is low to a lower level where the soil is quite as soft but has a higher carrying capacity by virtue of its depth, as previously explained.

It has been my privilege to be associated with the firm of Messrs. Palmer & Turner, of Shanghai, in the construction of the Hong Kong Bank, the Chartered Bank, and the Yokohama Specie Bank, all of which are treated in this manner, and at the present time on the new Custom House. As the foundations are similar in all these cases, it will be sufficient to describe one of them, and for this purpose I select the Custom House.

This building is 450 ft. by 140 ft. in area, and has normally 7 floors, though the front block has 9, and has in addition a tower 265 ft. high (see *Fig.* 9). Drawings are shown of the raft and the piles (*Figs.* 10 to 14). The raft takes the form of a continuous slab of reinforced concrete varying in thickness from 16 in. to 24 in. under the tower and stiffened by ribs in both directions, which are generally about 22 ft. apart. These ribs form reinforced concrete beams and normally have a depth of 6 ft., including the bottom slab, but under the tower increased to 9 ft. The columns rest on the tops of these beams at their intersections.

The most difficult portion in the case of the Custom House is that immediately under the tower. The total load which comes down in four small areas under the four corners of the tower is altogether 6,700 tons, and this load is distributed by the raft over an area of approximately 60 ft. sq. Of this total load 2,240 tons are carried by the direct action of the soil on the raft and 4,460 tons by the 225 concrete piles in this area. It will readily be understood that to transmit these heavy loads over this area requires very considerable strength, and the details of the reinforced concrete beams are perhaps of interest. The main reinforcements are 1¼ in. diameter, and the largest beams have 16 such rods in the tension side. The shear strength has to be very carefully considered with beams of this kind, and both bent-up rods and stirrups are used for this purpose. The piles are of reinforced concrete 50 ft. long, 16 in. square, and the surface is serrated to give additional friction resistance.

One of the difficulties on a soil of this kind is that even after taking all these precautions considerable settlements are experienced. These settlements are not found to be damaging, provided the whole building settles at the same speed and the same amount. To ensure this it is necessary to proportion the carrying capacity of the back of the raft to its particular load, so that no portion shall tend to settle more than any other. This is effected by so distributing the piles as to provide for each section of the raft exactly the additional resistance

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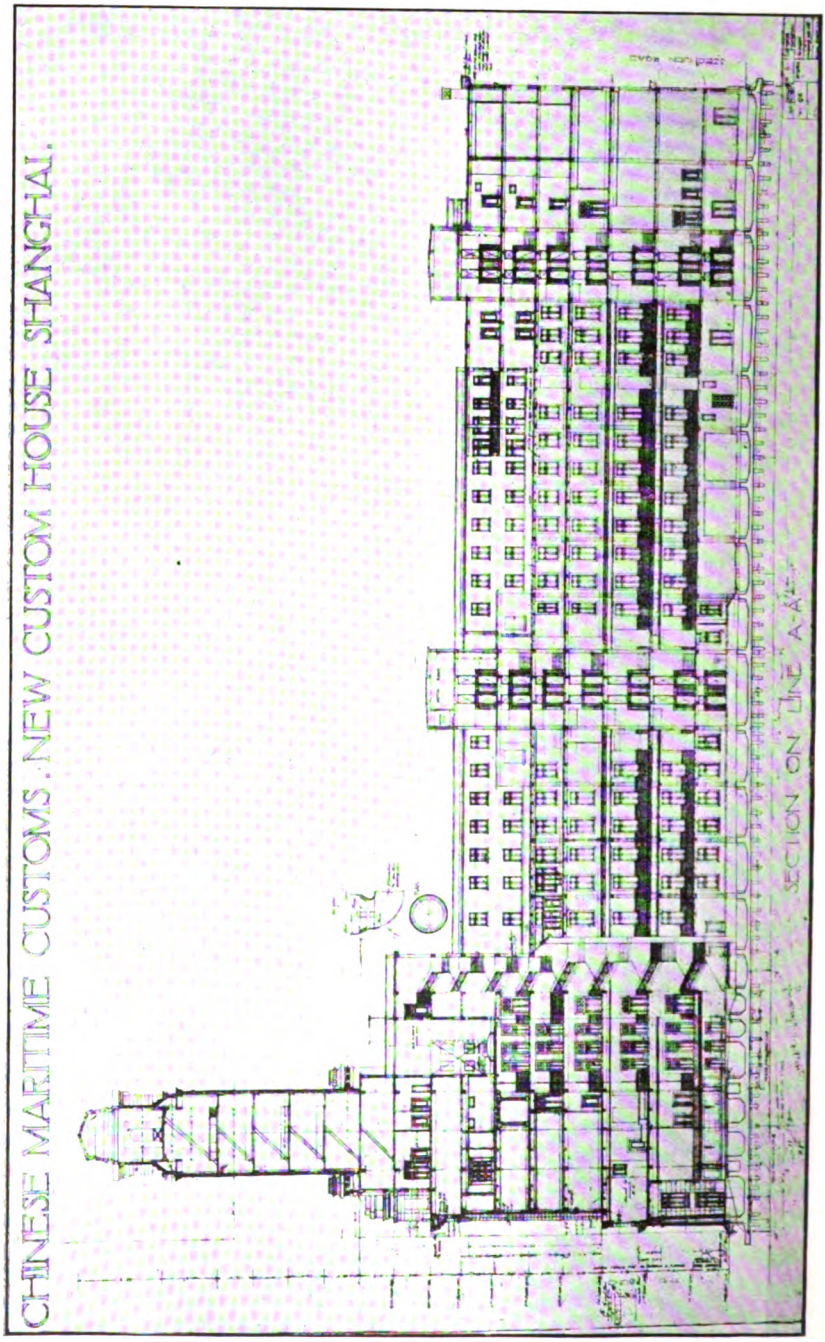

Fig. 9.—New Customs House, Shanghai : Longitudinal Section.

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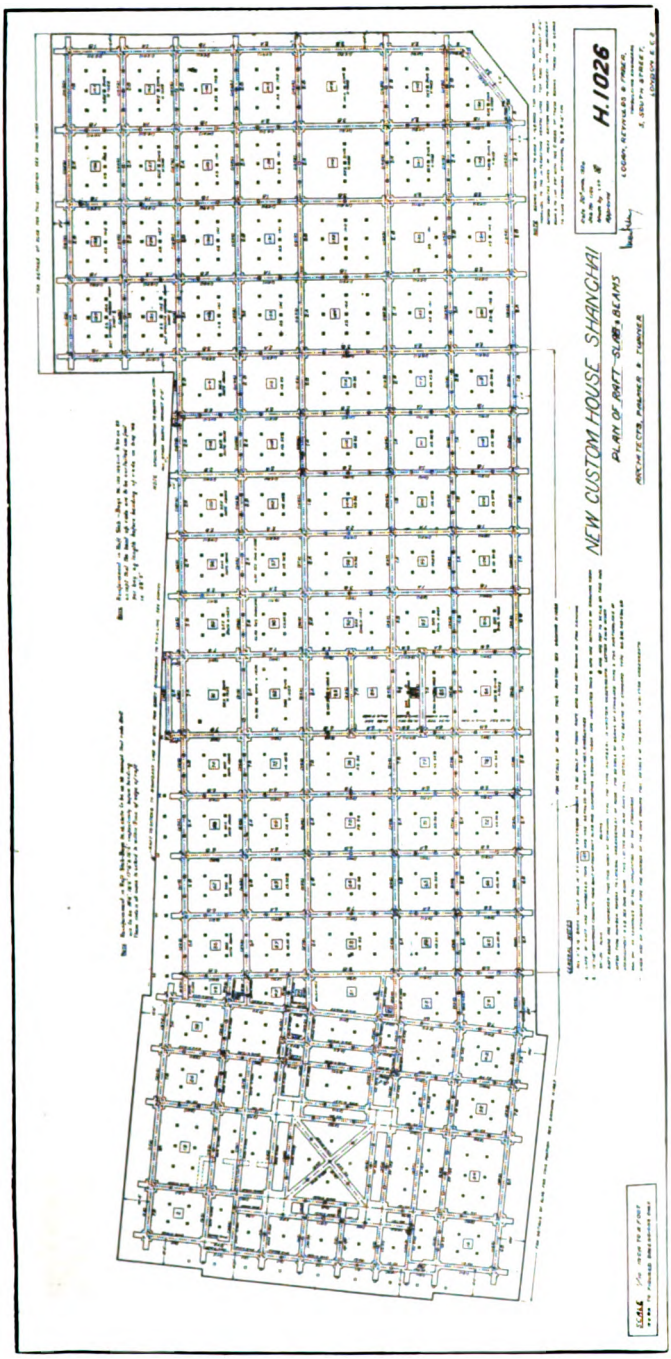

Fig. 10.—New Customs House, Shanghai : Plan of Raft Slab and Beams.

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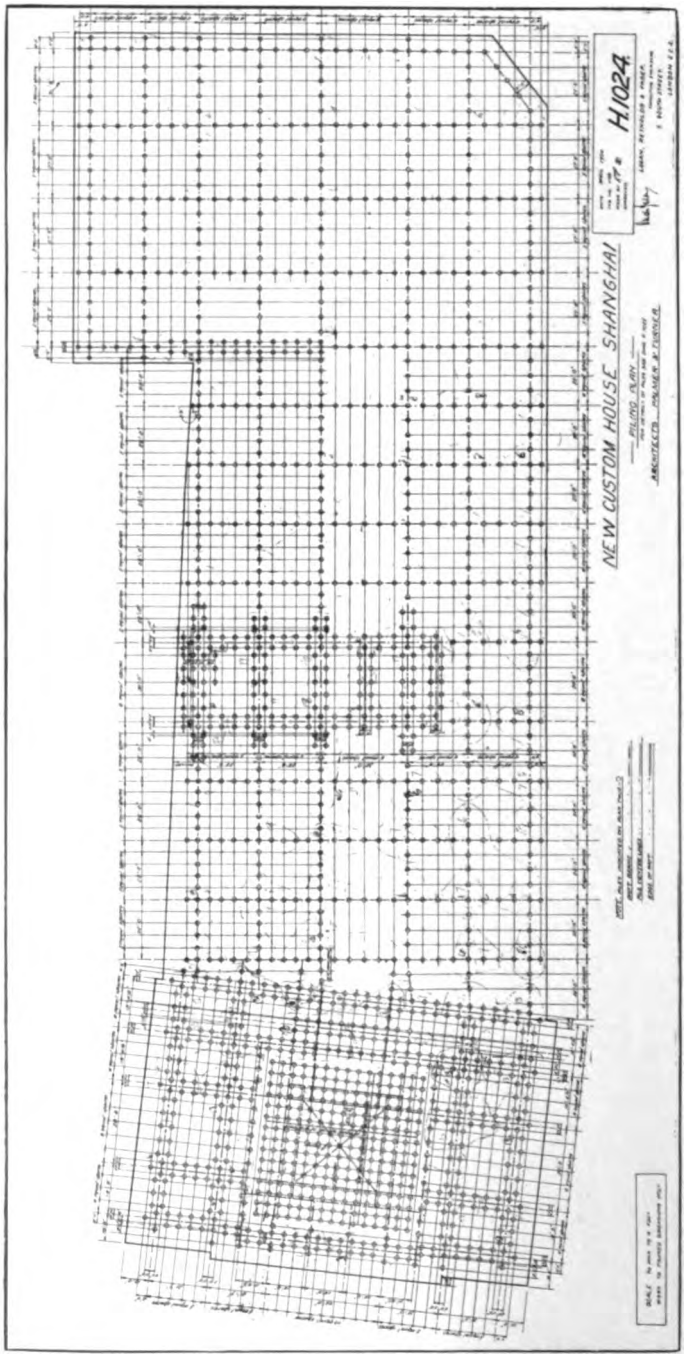

Fig. 11.—New Customs House, Shanghai : Piling Plan.

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required over and above what the soil itself will carry. In structures of this kind it is just as dangerous to make one part of the raft too strong as to make another part too weak, because the part that is too strong would fail to settle with the rest and would then cause the structure to break its back. The safe pressure on the soil allowed by the Shanghai regulations is only 1,600 lbs. per sq. ft., and it is remarkable that with such a soil buildings of this magnitude can be erected at all.

It is usual in the case of large buildings in Shanghai to estimate beforehand what the likely settlement will be and build the structure above its final level by this amount. In the case of the Hong Kong Bank and the Chartered Bank, the ground floor was made at a level of 12 in. above the position where it was thought likely to come to rest, and temporary steps were erected from the street, which were removed one at a time as settlement proceeded. The actual movement on the Hong Kong Bank up till last summer was approximately 9 in. to 10 in.

Concrete Piles.

Up till quite recently piles for carrying buildings were practically always of timber. This material has all sorts of advantages, but unfortunately one very serious drawback, viz. that even under favourable circumstances it cannot be relied on for more than 100 years, and where the timber is exposed to both air and water this period may be reduced to 20 years or even less. It therefore has serious disadvantages in modern building construction, and though I understand timber piles were recently used in a very large and important building in London I cannot imagine that this will be the practice of the future. The life of reinforced concrete is a subject about which there has been much speculation and difference of opinion, but I think the matter is fairly summed up as follows.

We have little actual experience of reinforced concrete of more than 30 years, but such as we have goes to show that with certain reservations it is as good to-day as it was the day it was made. These reservations are as follows :—

(1) Where the cover of concrete is inadequate in thickness, or where the concrete has been of faulty materials or gauged with insufficient cement, or where the concrete has been broken away by mechanical injury or was imperfect from the first, owing to carelessness in concreting, corrosion has set up.

(2) Reinforced concrete seems to stand fresh water perfectly, but suffers deterioration in sea-water and water in boggy or peaty land containing organic acids unless special precautions are taken. The difference between a design with adequate protection and one without, which may represent perhaps only 5 per cent. in cost, may mean the difference between 20 and 100 years of life. Nevertheless, the author is quite convinced, both from theoretical considerations and from the evidence of the oldest structures we have, that where the design and execution are the best of their respective kinds reinforced concrete in ordinary situations is the most lasting material which we have, with the possible exception of the best stonework and brickwork, the exclusive use of which is generally impossible in modern secular buildings owing to the excessive space which they demand.

In the case of concrete piles, we have an excellent application of reinforced concrete where it probably has no equal. One of its objections until quite recently was the length of time which it takes to harden sufficiently to stand driving. This disadvantage has recently been practically obviated by the great advances made in the manufacture of cement. Not only has there been a gradual but marked improvement in ordinary cement over the last 20 years, but remarkable advances in the last year or so are still more noteworthy. While the current British Standard Specification for cement still specifies 200 lbs. per sq. in. as the minimum tensile strength of 3 to 1 briquettes at the age of 7 days, it is now no uncommon thing to get samples of cement giving 350 up to 450 lbs. per sq. in. Early in December I received a sample which on test gave 548 lbs. per sq. in.

The Cement Marketing Company have recently brought out a new rapid hardening Portland cement which I had the opportunity of testing quite recently in which the tensile tests gave

312	lbs.	per sq.	in. in	1	day.
516	,,	,,	,,	2	days.
583	,,	,,	,,	3	,,
611	,,	,,	,,	5	,,
688	,,	,,	,,	7	,,

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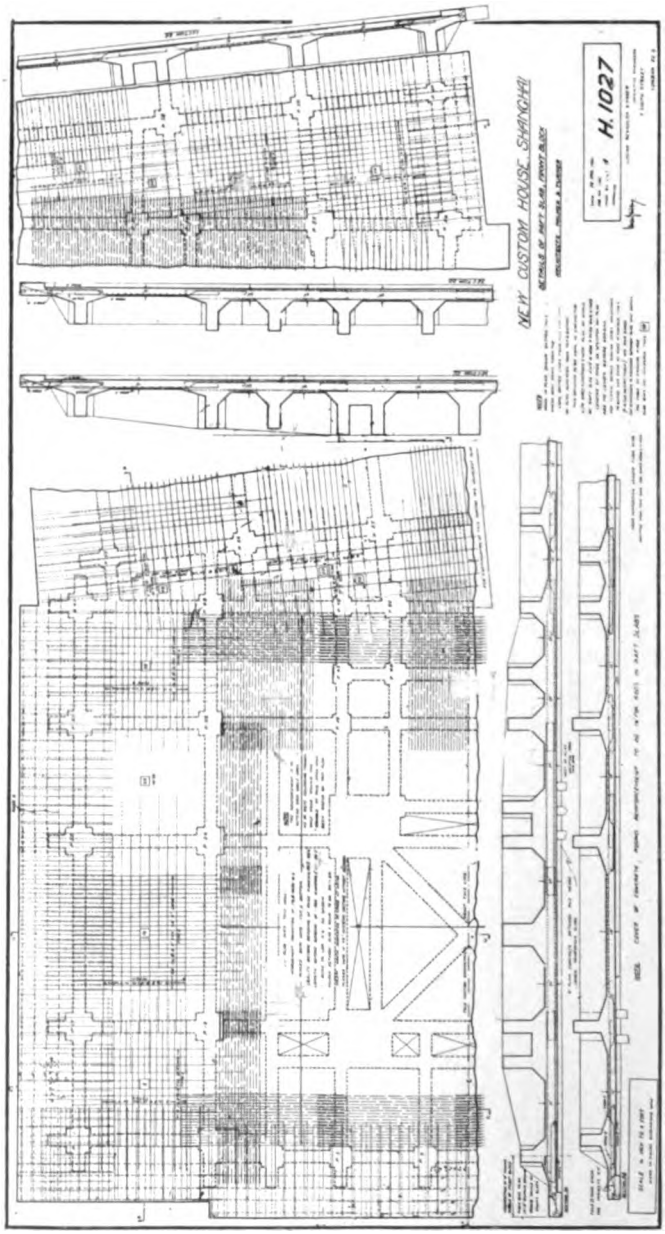

Fig. 12.—New Customs House, Shanghai : Details of Raft Slab, Front Block.

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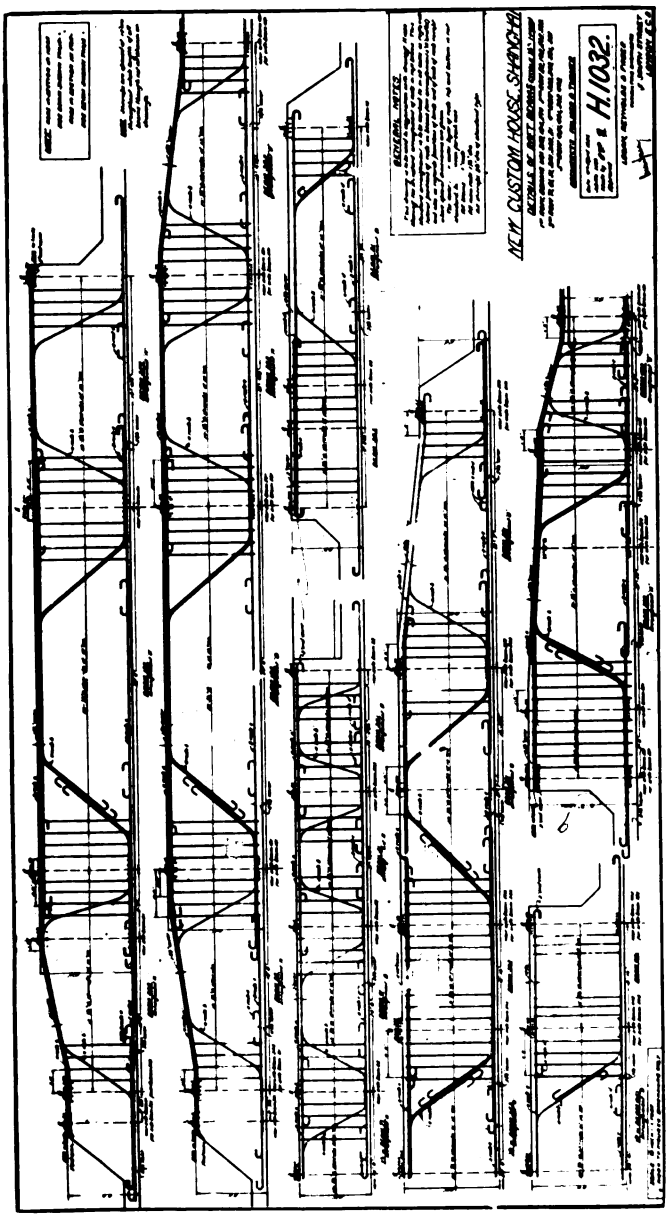

Fig. 13.—New Customs House, Shanghai : Details of Raft Beams.

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The corresponding crushing stress, showed the same advances over ordinary cement. It will be seen that this cement attains a greater strength in one day than the British Standard Specification would require as a minimum in 28 days. Six to 1 concrete made with this same cement gave the following stress in crushing :—

1 day	. . .	1,750 lbs. per sq. in.
2 days	. . .	2,910 ,, ,,
3 ,,	. . .	3,853 ,, ,,
5 ,,	. . .	4,840 ,, ,,
7 ,,	. . .	5,360 ,, ,,
28 ,,	. . .	6,810 ,, ,,

The effect of this is to give the strength in one or two days which we were previously satisfied to get at the end of 28 days, and the effect of this on reinforced concrete work is obvious. Compare this with ordinary concrete where a strength of about 2,000 lbs. per sq. in. in 28 days is quite normal and it will be realised what tremendous advance has recently been made. Advantage of this was recently taken by the author in a small work of considerable interest.

The foundations of Marlborough Hall were found to be settling when the structure was nearing completion and on examination it was found that cracks in the walls were opening and accurate levels indicated that considerable settlement was still taking place, an inch of settlement being recorded in a fortnight. The ground is water-logged immediately below the foundations, so that underpinning would have presented very great difficulties. It was also considered that to withdraw the water by pumping from the foundations it was desired to underpin would have produced a dangerous situation

The matter was dealt with by driving trestles consisting of two piles, one of which was driven as near to the foundation as possible and the other about 10 ft. away (see *Fig.* 15). Two 16 in. by 6 in. steel joists were then fixed to the tops of the piles, overhanging the nearest one so as to pick up the foundation. The nearer pile is then in compression and the outer pile acts as a tie to hold the end of the lever down. These piles ought obviously to be lasting, as the safety of the structure would depend on them ; it was not considered that timber would be suitable for this purpose, as the top of the timber piles would be alternately exposed to air and water, depending on the level of the river. Concrete piles were the ideal thing, but it was clear that we could not wait two months for the concrete piles to be made, matured and driven as the settlement in the interval would have endangered the building. Actually the concrete piles were made with the new aluminous cement and were successfully driven when they were two days old. They were driven with a 2-ton monkey with a 3-ft. drop to a set of about $\frac{1}{8}$ in. per blow. The steel cantilevers were then bolted down to them by straps and bolts through holes which had previously been cast in the piles to receive them, and the joists with their straps and bolts were then cased in concrete to protect them from corrosion. This has been entirely successful, and no movement has occurred since, and the work was done at a very moderate cost considering the circumstances.

While on the subject of concrete piles it is perhaps interesting to mention that there are in existence about ten formulæ for determining the safe carrying capacity of a pile from the weight of the monkey, its drop, and the final set per blow. These formulæ all give greatly differing results, and I have known cases where these results have differed as much as from 20 tons to 140 tons. It is clear they are all quite unreliable, and indeed they all fail to take into account some of the important factors on which the carrying capacity of the pile depends ; one of these, perhaps the most important, is the " taking up," as it is called, when the pile is allowed to rest after driving.

The Menin Gate in Ypres, for which Sir Reginald Blomfield is the architect, has foundations consisting of roughly 25 ft. of quicksand with blue clay below The foundation consists of about 500 ferro-concrete piles 40 ft. long extending through the quicksand and roughly 15 ft. into blue clay. A 3-ton monkey dropped 3 ft. produces on these piles a set of somewhere between $\frac{1}{4}$ to $\frac{3}{8}$ in. per blow. An interesting test was made on some of these piles by leaving them for a month or so and then applying the same blows to a pile as had produced $\frac{3}{8}$-in. set per blow during driving. The effect of 20 blows on the pile was nil and the pile absolutely refused to drive after a period of rest. This taking up depends very much on the nature of the soil, and is accounted for as follows :

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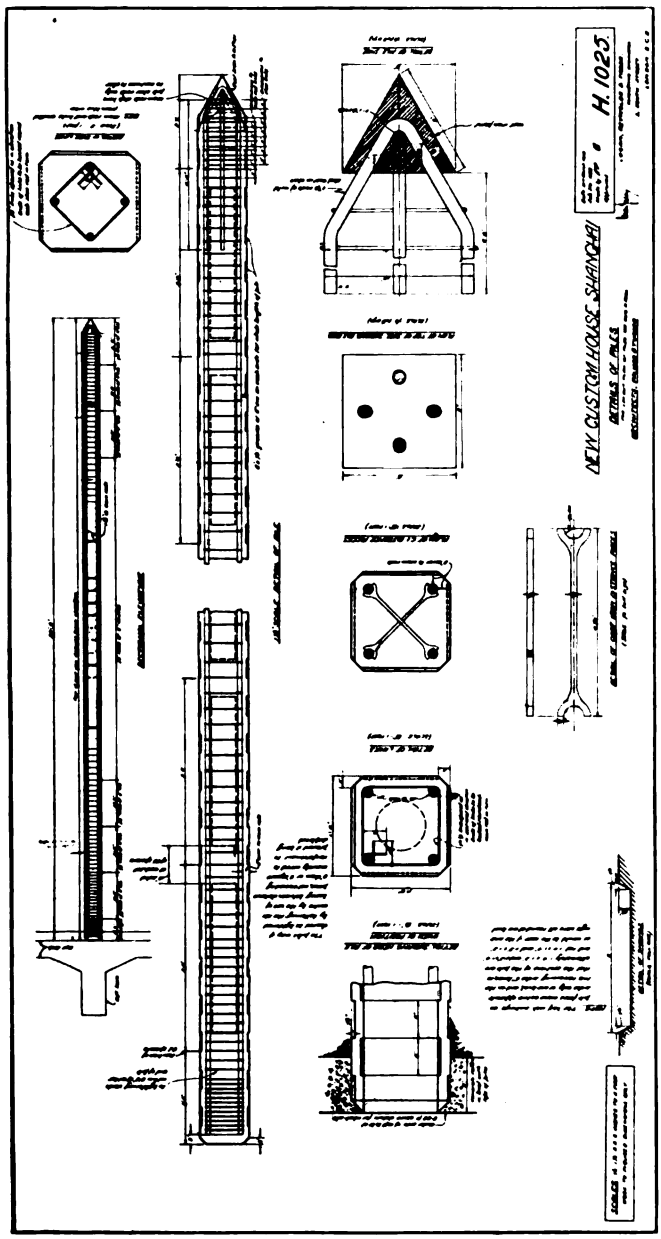

Fig. 14.—New Customs House, Shanghai: Details of Piles.

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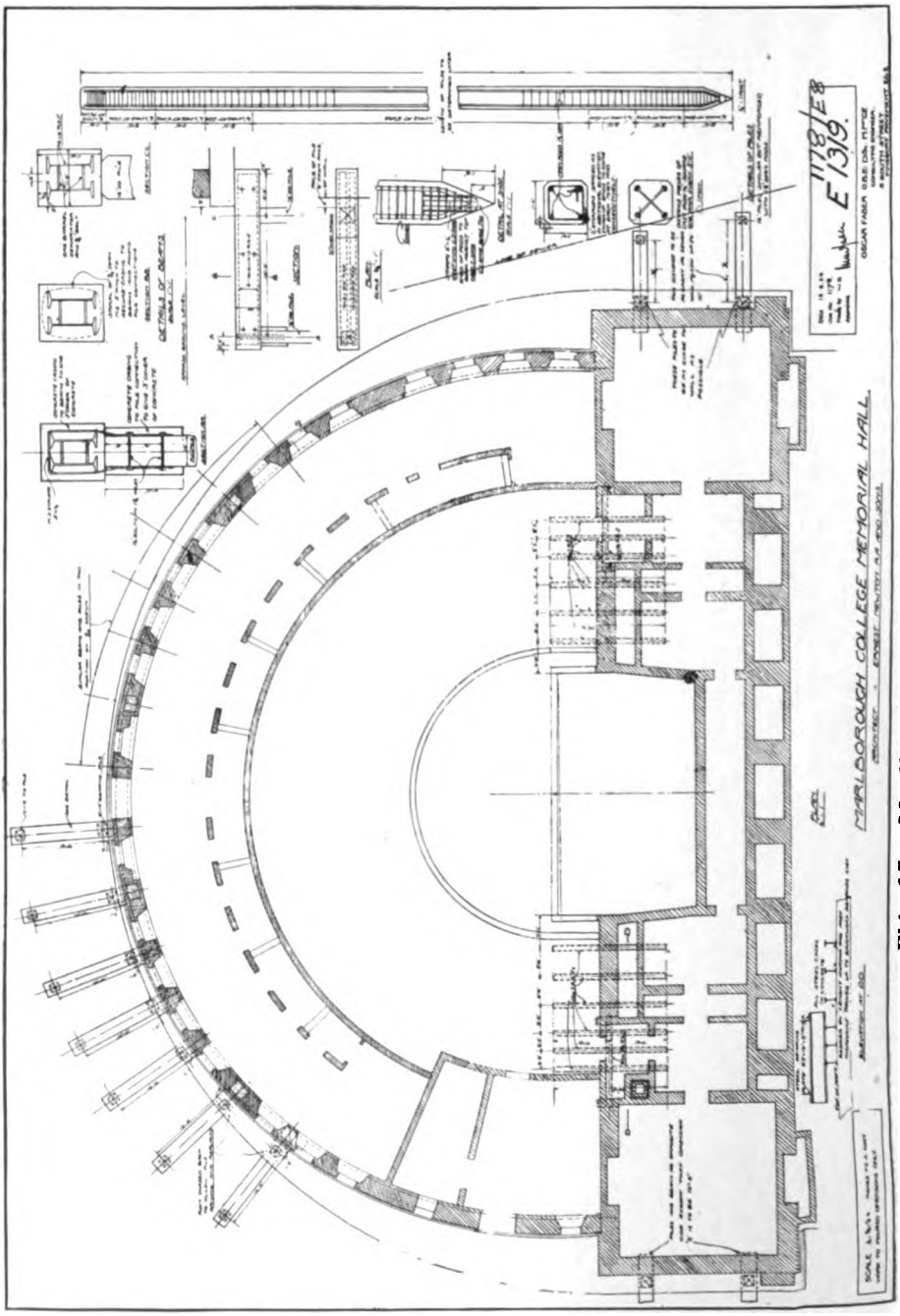

Fig. 15.—Marlborough College Memorial Hall.

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When the pile is first driven the soil immediately surrounding it has its cohesion shattered by the vibration of driving, and therefore driving proceeds comparatively easily. After a period of rest the soil is pressed against the pile owing to the hydrostatic pressure existing in the soil, and this cohesion is gradually built up again. In other words, the pile sticks to the clay or surrounding earth and this cohesion has to be broken down before further movement of the pile will take place. The whole of this cohesion is available for supporting the pile and is a very large element of the total resistance. This additional cohesion, or "take-up," is greatest for blue clay and least for dry sand, and therefore any pile formula which fails to take into account the nature of the material through which the pile is driving can obviously only be misleading.

The author has found it much better to pre-determine the length of a pile and its carrying capacity from a consideration of the skin friction or cohesion between the pile and the soil, which with a little experience can be estimated for various soils with considerable accuracy. Where time allows an actual loading test of a group of 4 piles is highly desirable on important structures, and is the only reliable criterion of their carrying capacity.

Where it is desired to carry foundations down through a waterlogged soil there are in general three good methods. The first is by using bearing piles as already described. If the foundation at the bottom consists of clay or other impervious material into which steel piles can be driven sufficiently to make a water-tight joint, then a second good method is to drive sheet steel piles all round the foundation and a few feet into the stiff clay at the bottom, when the excavation can be removed, and concrete placed in the ordinary way.

Where, however, the foundation is rocky and a water-tight joint cannot be made by sheet piles, then caissons generally provide the best solution. A caisson consists of a box composed either of steel plates or of reinforced concrete, which is gradually sunk into the ground by weighting it. The caisson has a cutting edge at the bottom and a horizontal diaphragm some 8 ft. above the cutting edge. Access to the working chamber below the diaphragm is obtained through tubes extending from the diaphragm to the top and provided with air-locking arrangements. Men are lowered down into the working chamber, from which water is excluded by means of compressed air. The soil is then dug out from under the cutting edge, and sent up to the top in skips through the air-locking tubes. As the caisson sinks additional sections are built on to it on top. When the rock is reached its surface is cleaned up, and concrete is then placed from above so as completely to fill both the working chamber and the space above it.

The Federal Reserve Bank in New York had its outer walls constructed of reinforced concrete caissons in this manner, the caissons being 30 feet long and 8 feet wide. A special feature in connection with this work is that the caissons were lowered down 100 ft. on to rock, although the site is bounded by narrow streets with tall and heavy buildings on the other side, which were not disturbed or underpinned during these operations.

In conclusion I would like to refer briefly to a special application of reinforced concrete to the strengthening of a brick retaining wall. A large site had been excavated to a depth of 30 ft. approximately, and the surrounding earth, together with the buildings upon it, was being retained by means of a brick wall 3 ft. in thickness. At the time when the author was called in the wall was showing signs of overturning owing to the pressure of earth behind it. It was immediately shored to render it safe, and subsequently has been strengthened.

In connection with this work the author formed his opinion, after an examination of the soil, of the pressure on the back of the wall, but desired, for several reasons, to have a check of these pressures if possible, and it was found that the trench for the retaining wall was strutted across by means of horizontal timber struts 7 ft. apart in both directions, the sectional area of the struts varying between 7 in. sq. and 8 in. sq. This, of course, was on those portions of the wall where the dumpling had not yet been removed, and where the struts were still resisting the full horizontal pressure corresponding to their particular depth. It was noticed that the struts

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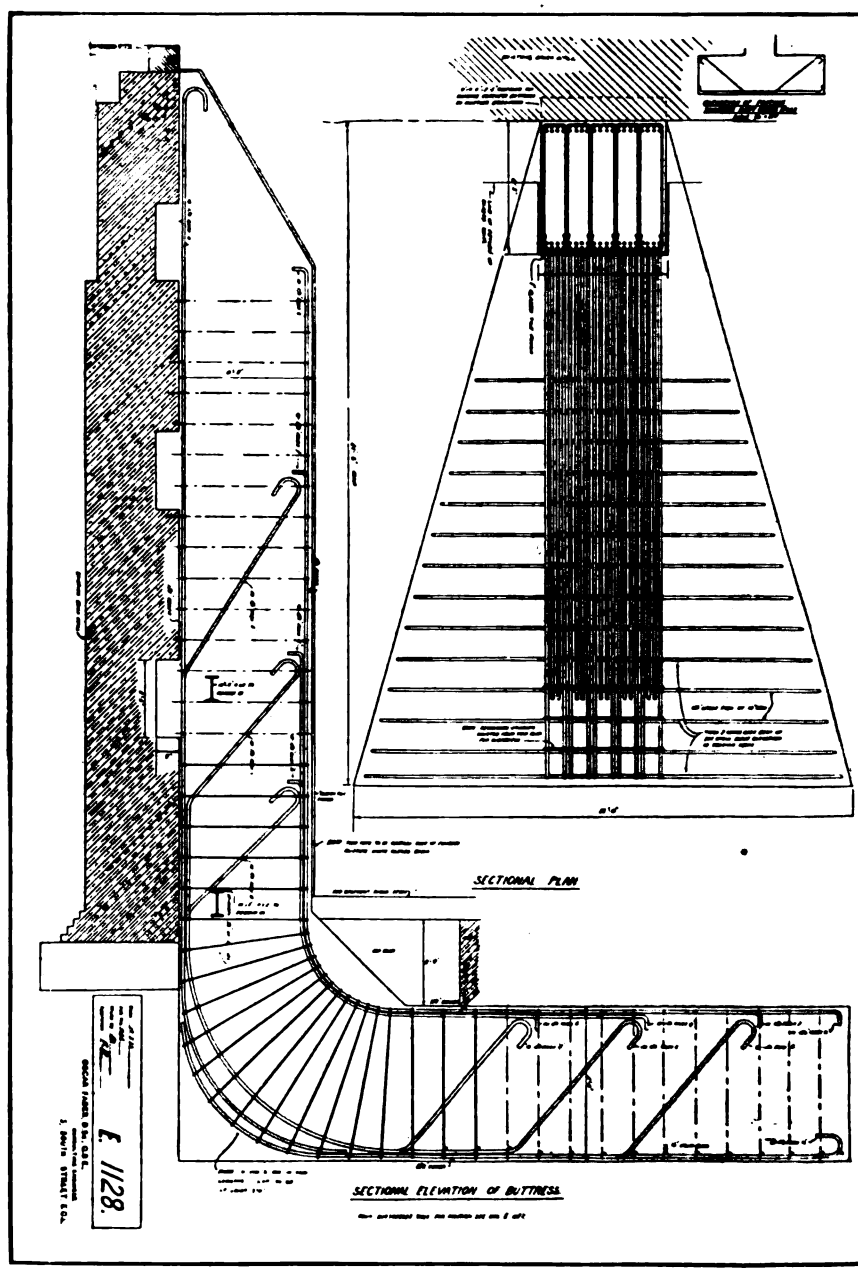

Fig. 16.

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had indented themselves to a considerable distance into the horizontal timber walings, and this suggested the means of checking the earth pressure. A particular strut half-way down the wall was found to be 7½ in. sq. and the indention to the horizontal timber, which amounted to about ⅜ in., was measured. A similar strut and a similar piece of waling were then tested in a testing machine, and it was found that a pressure of about 28,000 lb. was required to produce the same indention of the strut into the waling.

Having regard to the fact that these struts were 7 ft. apart in both directions, it was possible to check back and calculate that the earth pressure at this depth must be of the order of 28,000 ÷ 49 = 570 lb. per sq. ft. The calculated pressure at that depth by the author's assumed pressure curve came to 630 lb. per sq. ft., which was considered to be in close agreement. This was considered of sufficient interest to be recorded, because it is not often that we have opportunities of checking the pressures for which retaining walls need to be designed.

Fig. 16 shows how the walls were actually strengthened. A reinforced concrete buttress of rather peculiar design was constructed against the inside of the wall at intervals and efficiently bonded to the existing brick walls. Its effect is to increase the leverage of the weight of the wall from about 1 ft. to about 15 ft., and is obviously extremely effective in supporting the wall against any tendency to overturn. It will be readily appreciated how extremely economical this method of strengthening the wall was as compared, for example, with the construction of a continuous reinforced concrete wall inside the brick one. From the point of view of obstruction, the arrangement adopted also proved to be peculiarly convenient, as these buttresses could be placed in certain recesses in the brick wall. It will be noticed that the buttresses are heavily reinforced, as might be expected, as the pressure on a considerable width of wall was being resisted by a buttress only 4 ft. wide, and it was necessary to reduce the dimensions to an absolute minimum with a view to causing as little obstruction as possible. Actually the composite wall consisting of the ferro-concrete buttress with the brick wall between the buttress is no more expensive than the most economical type of brick or reinforced concrete retaining wall which would be safe to resist the actual pressures experienced.

Concrete-Block Walling Cast *in Situ.*

A SYSTEM of building concrete walls of blocks cast in position has been patented by Mr. A. H. Barnes, of 14, Sydenham Rd., Croydon, and is illustrated on the following page. With the exception of the bottom and top course, the blocks are T-shape, as shown in the elevational drawing in *Fig.* 1. The first course consists of plain rectangular blocks laid so that there are gaps between them of the same length as the width of the vertical leg of the T-block. The next and subsequent courses are formed of the T-blocks, as shown in *Fig.* 1, the vertical legs of the T-blocks filling the spaces between the blocks which form the course immediately below.

The mould in which the T-blocks are made (*Fig.* 2) consists of four longitudinal members of angle steel and four transverse lateral members passing through slots in them. In these lateral members are holes for adjusting the shuttering to various thicknesses of walls by engaging them with catches on the longitudinal members. On releasing the pins which tighten the side and end members of the mould together the sides and ends expand free from the work and the mould can be removed without being taken apart. These longitudinal and transverse steel angles carry the boards or metal sheets forming the shutters.

Alignment of the wall is secured by the lower portions of the moulds bearing against the blocks in the lower courses. It will be seen that no special blocks or moulds are required to form a wall of any definite length, as the variation is obtained by lengthening or shortening the mould. Similarly, the forms can be adapted for any thickness of wall, and it is a simple matter to form a cavity wall by inserting sheets within the mould.

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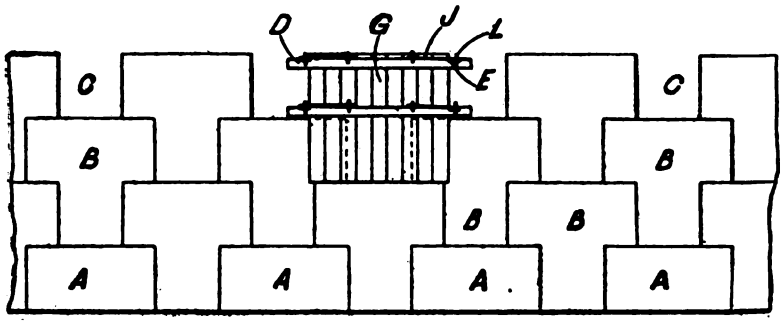

Fig. 1.

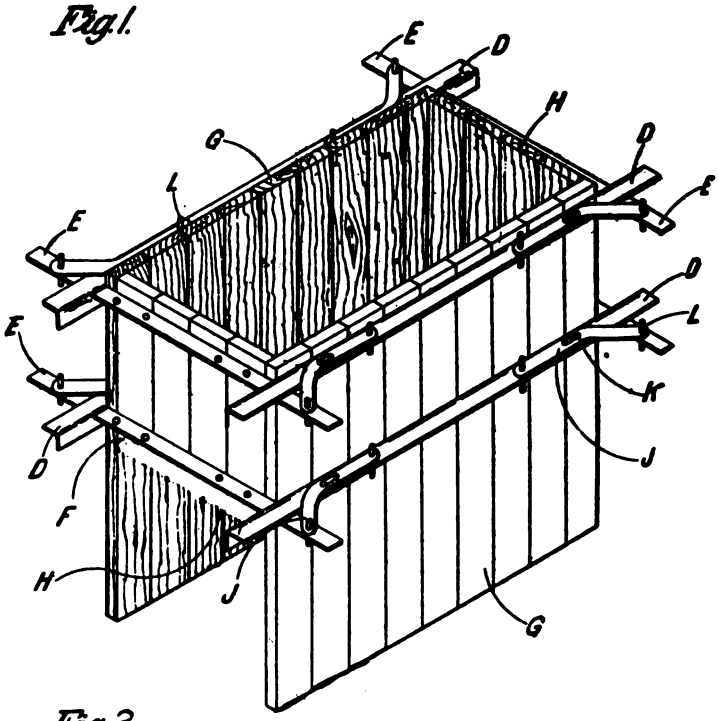

Fig. 2.

Concrete-Block Walling Cast In Situ. (*See p.* 455).

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Design of Formwork for Reinforced Concrete Construction.

By A. E. Wynn, B.Sc., A.M.Am.Soc.C.E.

IX.—DETAIL CONSTRUCTION OF BEAM AND GIRDER FLOOR FORMS.

THE method of calculating the size of timbers required for the several parts of a beam and girder floor has been shown in the previous chapters, so it will only be necessary now to describe the methods of framing and stripping. There is only one general method of framing, but there are several ways of constructing details to facilitate stripping.

The method of constructing the details will determine whether the formwork is economical or expensive. The main thought to bear in mind when designing the forms is the ease and sequence of stripping. A form is only temporary, and it must be built so that it can be taken apart with the least cost and the maximum salvage of material. The speed required on the job, and, closely allied to this, the need for reshoring the beams, will also affect the details of erection.

It may be thought by some that reinforced concrete building is slow because of the time required for the concrete to set. This is not true, however; with proper attention paid to details of formwork and reshoring of beams a reinforced concrete building can be put up as fast, if not faster, than any other type of building. It is common practice in the United States to complete a floor every week or ten days, whatever the size of the building may be.

In a multiple-story building it is customary to build only one complete set of forms for one floor, or for a floor and a half, and use these again on the upper floors, so that it is necessary to strip as soon as possible, and the forms must be designed so that the least important parts can be stripped first without disturbing those members which carry most of the load.

In beam and girder construction the unit method is always used, that is, the slab panels, beam and girder sides and bottoms are built up in advance to the correct size ready for erection, often going a step farther and building up the beam and girder boxes complete. A power saw is a necessity for economy, as there will be many small pieces of the same size to saw.

Timber sizes for the different parts will be practically the same for all jobs, except that the joist size may vary somewhat with the span. Before deciding on the joist to use, comparison for economy in material should be made between two or three different sizes. The standard sizes used for all ordinary buildings are as follows:—

For slab panels, 1 in. by 6 in. dressed to $\frac{7}{8}$ in. by $5\frac{1}{2}$ in. tongued and grooved.
,, joists, 2 in. by 6 in., dressed or rough.
,, beam ledgers, 1 in. or $1\frac{1}{2}$ by 4 in. or 6 in., dressed to $\frac{7}{8}$ in. or $1\frac{1}{4}$ in.

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For cleats or battens, 1 in., 1½ in. or 2 in. by 4 in. dressed.
,, beam and girder bottoms, 2 in. plank dressed four sides.
,, ,, ,, ,, sides, 1 in. by 6 in. dressed if only used once and 1½ in. by 6 in. dressed if used more than once.
,, posts, 3 in. or 4 in. by 4 in. dressed or rough.

In choosing between dressed and rough timber, faces coming in contact with the concrete should always be dressed, and it is customary to order material for slab sheathing, joists, beam sides and bottoms to be dressed four sides. Beam bottoms may be dressed on one side only if thereby a single plank, or two planks together, will give just the required width of beam. Joists can be rough, but are harder to handle and the ends will have to be sized to an even depth. Posts are generally rough and are stronger than when dressed, but if they can be used elsewhere, as for studs, they should be dressed.

The bill of material should be made out from sketches of the forms, and not ordered haphazard. Required lengths particularly should be noted, otherwise there will be an astonishing percentage of waste. The sketches for the different parts should be given to the carpenter foreman so that he will know for what part the different items were ordered.

If there is a basement wall to build, the timber can be used again in the floors and should be ordered with that in mind; that is, it may be cheaper to use 3 in. by 4 in. or 2 in. by 6 in. studs in the wall and re-use them as posts or joists in the floor rather than order 2 in. by 4 in. studs and have no use for them afterwards.

As the various parts are framed up as units it will be convenient to consider them separately before describing the method of erection and joint details.

Slab Panels.—The average size of a slab panel between beams and girders will be about 5 ft. by 18 ft. This can be built in one, two, or four sections, the size depending mainly on the facilities for handling from floor to floor. With a hoisting engine on the job one panel would be the most economical, but if they are to be handled by hand two or four panels per bay would be better.

The panel will rest on the top of the beam and girder sides, and should be ½ in. to 1 in. less in width and length than the clear dimensions between the concrete. They will thus set only partly on the sides, allowing ¼ in. to ½ in. clearance all around for "give" in the top of the sides—and they will always give slightly under the pressure of the concrete—and so prevent the panel binding in the concrete and hence being difficult or impossible to strip. The edges all around should be bevelled to 45 degrees.

The boards are nailed together with 1 in. by 4 in. battens spaced the same distance apart as the joists will be, and so that they will be about midway between joists. The battens must be about 2 in. shorter than the width of the panel. As panels are liable to swell if left long exposed to rain, to prevent buckling an end board is often left off until the last minute; or if two panels are placed side by side a narrow strip is left

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between them afterwards filled in or covered with a strip of tin or heavy paper.

Another method of construction is to nail the boards direct to the joists, using them as battens. This will save material and labour but it makes the panels heavier to handle, so should only be used when there is sufficient equipment to handle them. The panels are made in one section for each bay. Each end board is nailed on lightly for easy stripping, since if they stick they will remain in place while the rest of the panel can be removed. Sometimes the length of the panel is made a board-width short each end and these boards nailed on lightly to a joist placed a board-width away from the girder, for the same purpose of easy stripping. One of the end boards is left off until last to allow for swelling and to provide an opening for cleaning out the forms.

If the panels are to be used only once, unless it is desired to make them up in advance, loose boards can be used without the battens.

Joists or Spreaders.—The length of the joists will be the clear distance between beams less twice the beam side thickness less ½ in. The ends should be bevelled inwards at the bottom about ¼ in. so that they can be easily stripped without binding. If rough joists are used the ends must be sized to an even depth to give a level slab. When they are nailed direct to the sheathing the length and bevel will be the same.

Beam and Girder Bottoms.—The width of the plank should preferably be the same as that of the beam so that cleats will be unnecessary, but, as the cost increases with the width, above about 10 in. wide it is cheaper to use two planks nailed together with 1 in. by 4 in. cleats on the underside, 24 in. to 30 in. apart. The cleats must not be longer than the width of the bottom.

The length of the bottom will depend on whether it is to be carried on the column sheathing or blocked up from the yokes and butted against the sheathing. In the former case the length will be the clear span between columns less ¼ in. to ½ in. clearance at each end, and the ends will be bevelled inwards at the top. In the latter case the ends will be square, and the length will be the clear span between columns less twice the column sheathing thickness. In the first case a little leeway in measurement is allowed, but in the latter the measurement of the bottom must be accurate. The former method is generally used when the bottoms are to remain in place after the sides are stripped and the latter when the sides and bottom are stripped together as a unit, as it gives a little more clearance after the columns are stripped. When the bottom is carried on the sheathing the column is a little easier to strip. When the beam bottoms are carried on the girders either construction may be used and the length will be accordingly, the girder taking the place of the column.

Girder Sides.—The depth of the side will be measured from the underside of the slab sheathing to the bottom of the girder bottom. The side is usually made exactly this depth, but to save ripping a board all full-width boards may be used and the side allowed to project below the bottom, in which case notches will have to be cut at the posts to

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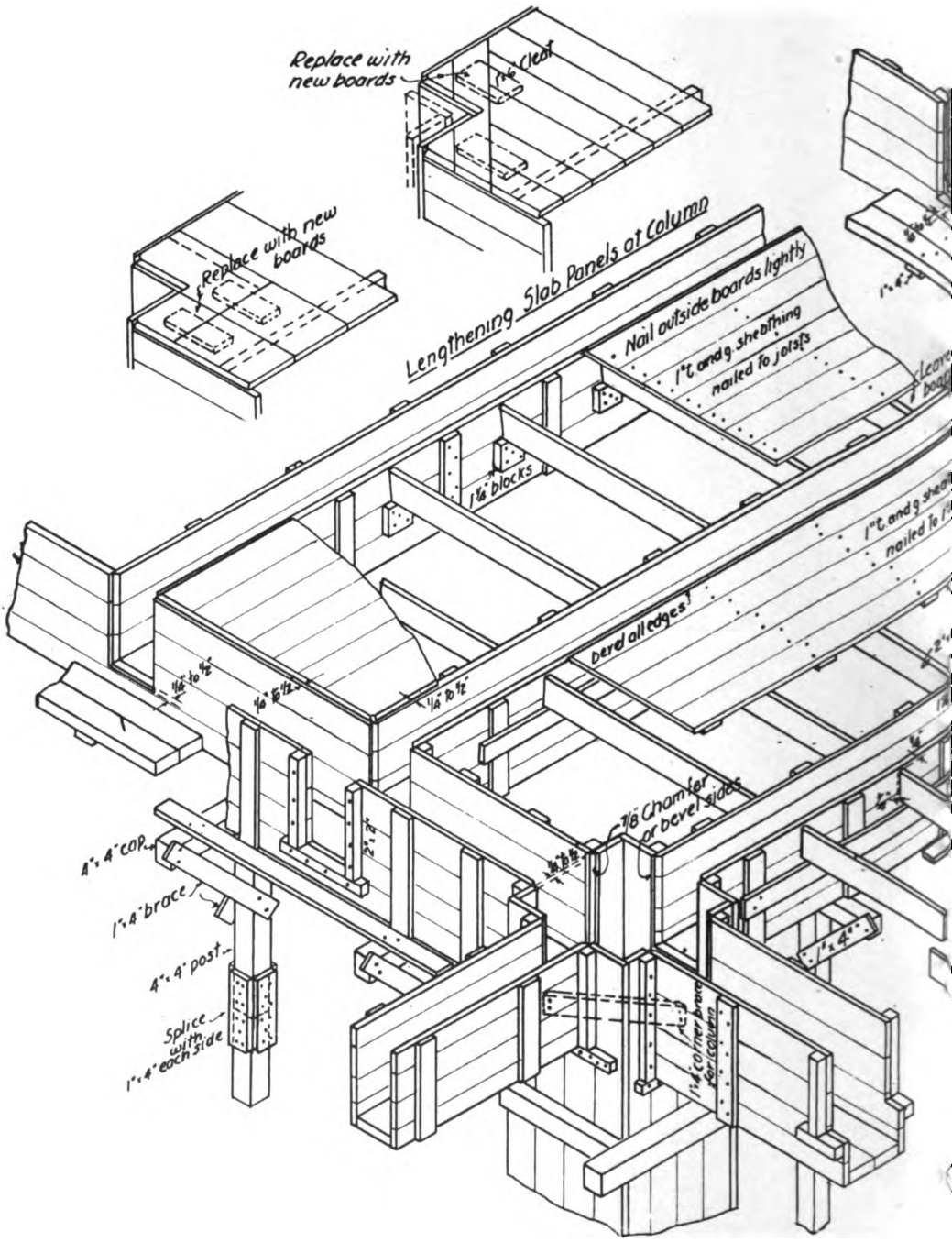

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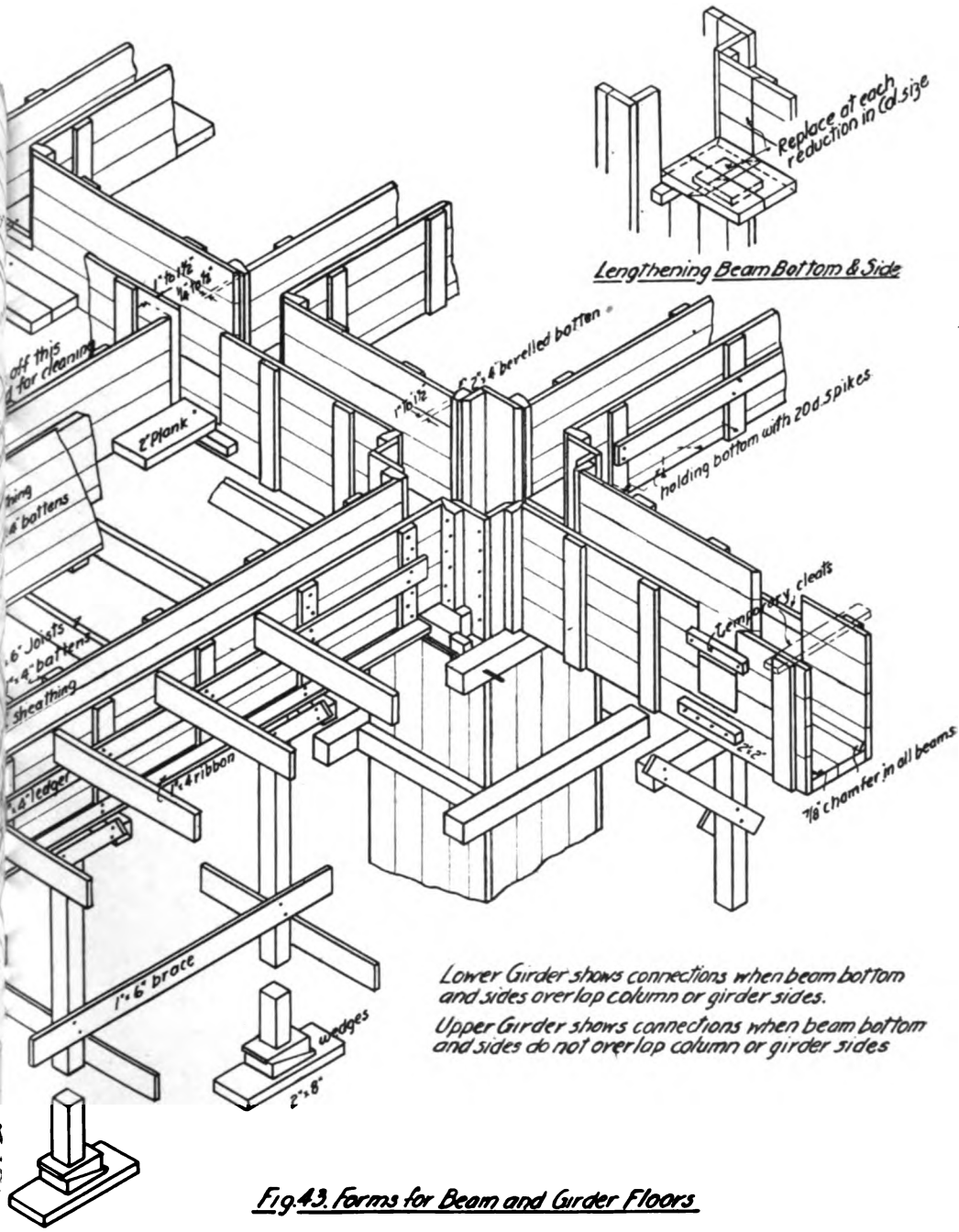

Fig. 43. Forms for Beam and Girder Floors

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allow the bottoms to bear on the caps. The sides should never bear on the bottom, but always overlap. The length of the side will depend on the system of framing used.

The best method is to make the length of the side equal to the clear span between columns less twice the thickness of the column sheathing less about 1½ in. each end for clearance; that is, if the columns are 18 in. square and 18 ft. on centre and the sheathing used is 1¼ in. the girder side will be 16 ft. 0½ in. long. The ends are bevelled outwards and the 1½ in. clearance is afterwards filled in with a bevelled batten, as mentioned later. This method is used with a column head as shown at (*b*) *Fig.* 12.

Another method is to make the length equal to the clear span less twice the sheathing thickness, or the ends of the sides will be flush with the column sheathing and are covered with an independent bevel strip. Still another method is to carry the sides over the column sheathing but stopping ¼ in. to ½ in. from the inside face and bevelling the ends inwards; in this case the length will be the clear span less ½ in. to 1 in.

The latter two methods, used with column heads as shown at (*c*), *Fig.* 12, make the column sides less easy to strip and leave small fins on the concrete at the junction of side and bottom bevels unless the space is filled in. The first method will be considered best practice, and the slightly extra labour cost will be more than saved when it comes to stripping.

The boards are nailed together with battens 24 in. to 30 in. on centre, the size depending on the depth, using 1 in. by 4 in. battens up to 15 in.; 1½ in. by 4 in. from 15 in. to 24 in.; 2 in. by 4 in. from 24 in. to 36 in.; and 3 in. by 4 in. for greater depths. The boards should be well clamped together before nailing, to close up the joints. The end battens should be a few inches in from the ends.

The openings for the beams correspond to the openings in column heads for the girders, and can be made by either of the methods shown in Chapter 7, and in *Fig.* 43. The former is the better; the width will be ½ in. to 1 in. wider than the width of the beam, the depth will be ¼ in. to ½ in. greater than the depth of the beam, and all edges will be bevelled inwards. If the beam bottom is carried on the sheathing, as must be done anyway if there is not much difference in depth between the beam and girder, the opening must be the depth of the beam plus beam bottom and the bottom edge will be left square; and the width of the opening will be that of the beam plus twice the beam side thickness. To support the bottom a short piece of 2 in. by 2 in. or 4 in. is nailed under the opening in either case. Temporary cleats should be nailed across the openings until the sides are erected and braced.

If the beams are deep, the openings will leave the girder sides weak, and they will be liable to break when stripping. A good method of overcoming this is to build the sides up to the underside of the beam bottoms as a unit with the girder bottom, nailing them permanently together and stripping together. The remaining portion of the sides is then built as separate units from column to beam and beam to beam,

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leaving 1½ in. clearances at each end if desired, and letting the battens project below the sheathing for lap (*Fig.* 44).

Beam Sides.—These are built the same way as girder sides, preferably leaving 1½ in. clearance each end as mentioned. The connection to the girder is the same as the connection of the girder to the column.

Beam Ledgers.—These are nailed on to the side battens at a distance down equal to the depth of the joists. They should stop off a few inches from the ends of the sides. The top surface must be level so that the joists will not have to be wedged up or cut down to give a level floor. For spans up to 6 ft., 1 in. by 4 in. will be sufficient; for greater spans 1½ in. material should be used. Instead of a continuous ledger, 6 in. by 6 in. blocks by 1½ in. thick are often nailed on the beam sides at the spacing of the joists. This will save some material, as waste boards can be used and the joists will be easier to strip.

Shores.—To the top of the post is spiked a cap of the same size as the post and about 15 in. to 18 in. longer than the width of the beam. The ends of the cap are diagonally braced back to the post by short pieces of 1 in. by 4 in. or 6 in. on opposite sides. The posts should be carefully cut off square at each end to obtain full bearing. The posts will rest on wedges, which should be hardwood, 8 in. to 10 in. long, 4 in. to 6 in. wide, and tapering from 3 in. down to about ½ in. The wedges are generally placed on a 2 in. by 8 in. sill about 12 in. long. If bearing on soft ground the size of the sill should be calculated to give a low pressure on the soil, not exceeding 1 ton per sq. ft. The required length of post should be noted before ordering, as this is a place where there can be much waste. Spliced posts should be avoided, but they are sometimes necessary, especially when a story height changes. They should have long cleats at the joint on all four sides, well nailed, and the abutting ends of the posts should be square.

The posts, if longer than 8 ft. will have to be braced at the centre with 1 in. by 6 in. boards in both directions, placed horizontally.

Erection.—*Fig.* 43 shows the assembled forms and the different methods of building the units and making the connections. The different parts being ready to erect the carpenters should work in three gangs, one placing bottoms, another sides and the last one joists and slab panels.

The outside wall columns are first set and plumbed, and on these are placed the beam bottoms all around the building, two corner columns being set first to give the line. This establishes definite points to work from, and the carpenters work in towards the centre of the building, setting next the first interior row of columns and placing the girder bottoms from these to the outside columns, then taking the next row, and so on.

One 20*d*. spike is placed in each end of the bottom and not driven home until the spacing of the columns has been checked with the tape. A shore is put under the centre and a carpenter walks along the plank and drives a spike into it through the bottom, and then the remaining

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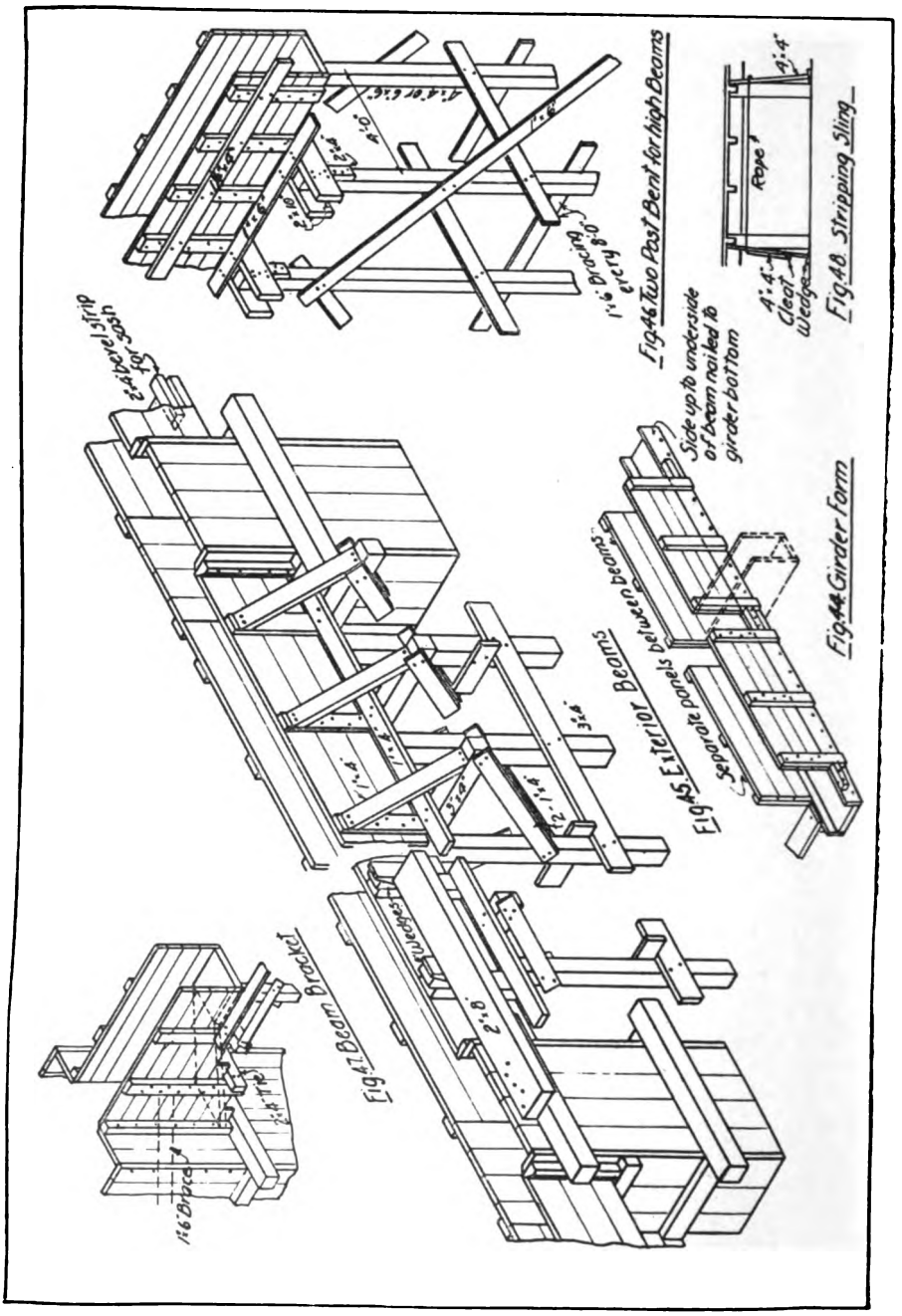

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shores are placed and spiked. The sills and wedges are inserted under the posts—they should not be placed at the top—and the wedges are driven tight so that the bottom has a camber at the centre of about ¼ in. in 10 ft.; this is particularly necessary when shoring from the ground, as there will always be some settlement.

If the girder bottom is not carried on the column sheathing, the top yoke should be placed at the right height to carry the bottom, and beam bottom is blocked up from this yoke.

The girder sides are lifted into place and lightly nailed to the bottom to hold them in place, and braced across the top to prevent spreading. The beam bottoms between girders are next placed and shored up. It is important that all shores are placed plumb to avoid eccentric loading, and hence bending in the shore; this is particularly true of spliced shores. The beam sides are then placed, lightly nailed and braced at the top; they should be set level. If 1½ in. clearance has been left, 2 in. by 4 in. battens bevelled on one side are now inserted and nailed to the column or girder. They are put on now instead of when making the sides so that the bevel can be cut to suit the exact opening. The centre of the girder sides should be braced back to the beam sides.

If the joists are not nailed to the floor panels they are now laid on the ledgers at the required spacing, leaving them loose. When they are levelled up the floor panels are dropped into place; these also are not nailed down, except for one or two nails at the corners to hold them in place. The panels must be notched out around the columns; this is more easily done in place than beforehand. The lower edges of the beams and girders are usually covered with a ⅞ in. chamfer strip nailed to the bottoms.

The bottom of the sides must be held against the pressure of the concrete, and this is done in various ways. Beams up to 24 in. total depth can be held by driving in a 20*d*. spike at each batten and two intermediate ones driven into the bottom on the slant and in opposite directions, leaving the heads projecting for easy stripping.

Without spiking the sides at all the best method is to nail a 1 in. by 4 in. or 6 in. ribbon on top of the shores on each side of the beam, and this will hold any ordinary beam. Sometimes the side battens are carried below the beam and tied across with short pieces of 2 in. by 4 in. below the bottom. Patent clamps are also used, but they are expensive, easily lost, and sometimes a carpenter forgets to tighten one up. Bolts above or below the bottom are also used, particularly when heavy battens are required; and it may be necessary to use a bolt or wires half-way up the beam if it is very deep, as then the side becomes similar to a wall form, even perhaps requiring a horizontal wale.

The last operation is to cross-brace the shores. Before commencing to pour, the wedges should be gone over and tightened up where necessary, and it is advisable to have a carpenter under the floor watching the forms, particularly the posts and wedges, all the time during concreting. Instead of erecting beam sides and bottoms separately, the beam boxes can be built as a unit; that is, sides and bottoms are nailed

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on the ground and erected and stripped together. One end of the box is up-ended to its position on the column, and the carpenters take the other end on their shoulders and with the aid of a post raise it up on to the top yoke of the other column. There will be more weight to handle, though fewer pieces, and the fitting can be done more accurately. The sides are spiked to the bottom as explained before, and ribbons are not used unless the beam is over 24 in. deep. The sides and bottom need not necessarily be stripped together, though they should be to use this method to the best advantage. It is generally economical for a one-story building, since the beams are light and the forms can be left in until they can all be stripped at the same time. In a multiple-story building, however, speed will generally require the sides to be stripped before the bottoms, so they must be independent units and half the advantage of this method is lost.

Nailing the joists to the floor panels should be done where it is possible to handle the panels, because there will only be one unit to place instead of several. Isolated beams with no slabs must be held at the top as well as at the bottom, either by wood ties, bolts, or wires. It will be noticed that few nails are required during erection; where they have to be withdrawn double-headed nails are an advantage. All these details are shown in *Fig.* 43.

Exterior Beams (*Fig.* 45).—Exterior beams along the building line and interior beams around a stair opening or elevator will have no slab on one side to brace the top, so some other means must be used. The commonest method is to make the post caps longer on the outside and run 1 in. by 4 in. diagonal braces from the caps to the top of the sides, preferably bearing against blocks to prevent slipping. It is a little more difficult to align the side with this than with the following method, and also more posts are required.

Instead, a 2 in. plank (scaffold plank will do) can be spiked to the top column yokes of adjoining columns and the sides held to line by driving wedges between the plank and side. It is then easy to rectify the alignment if required by tightening or loosening the wedges. The bottom of the side is held as before with a ribbon or spikes. Posts should be placed at least as close as with interior beams, although the load is less; and they should have firm bearing, since settlement will look bad and interfere with the setting of the sash. No camber should be given the beam if a sash frames into it. A bevelled 2 in. by 4 in. must be placed in the bottom to form a slot for the sash or as a nailing strip for a wood frame.

(*To be continued*).

Concrete Lumber City.

In order to prevent the spread of disastrous fires, the lumber city of Longview (Washington) has been built entirely of concrete. The work, which has just been completed, occupied two years.

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The Foreman's Guide to Concrete.—X.

A MANUAL FOR ALL ENGAGED IN CARRYING OUT CONCRETE AND REINFORCED CONCRETE STRUCTURES.

By Albert Lakeman, M.S.A., M.I.Struct.E.

REINFORCED CONCRETE WALLS.

REINFORCED concrete is used for the construction of walls of different types. The consideration of the thickness of the concrete and the amount of reinforcement to be provided will afford some scope for the designer, and every effort should be made to evolve an economical design. The walls can be divided into two main types, viz.: (*a*) the walls of a structure above ground, and (*b*) retaining walls for the support of earth or water.

In the first type the walls are usually in the nature of panel filling between the main weight-carrying members of the framework, and no great strength is required as the filling merely acts as a screen to keep out the weather. Generally speaking, these panels may be of any thickness not less than 4 in., and it is seldom that a greater thickness than 6 in. is required when the loads are all transmitted to the framework of the structure. The regulations of the L.C.C. state that each panel in any external wall shall be designed to resist safely a horizontal pressure of at least 30 lbs. per sq. ft., assumed to be acting uniformly over the area of one panel from either side. It will be observed that the panel must be strong enough to resist pressure outward as well as inward, and because of this the reinforcement must either be placed in the centre of the thickness of the concrete or the concrete must be reinforced in both inner and outer surfaces.

The panel becomes in theory a slab on edge in a vertical position with a horizontal thrust on either face, and all the points mentioned in connection with floor slabs will apply except that the dead weight of the slab itself will not have to be added to the applied load or pressure, because this dead weight will not cause any bending movement. Unless the distance between the main weight-carrying piers is excessive the specified pressure of 30 lbs. per sq. ft. will not produce a very large bending moment, and the theoretical thickness of concrete and the sectional area of steel required will be very small. Frequently the practical requirements will determine the materials necessary.

The pressure of 30 lbs. per sq. ft. is based on an allowance for wind pressure when considered as a force tending to push the panel inwards, and the outward pressure allowance is necessary to provide for the suction that occurs on the leeward side of a building during a gale and also to allow for pressure that may be caused by materials stacked against the wall inside the building.

Owing to the small amount of concrete frequently used in wall panels it is important that the reinforcement be accurately placed and maintained in position during concreting, otherwise there is a serious risk of some portion of the steel being exposed on the face of the wall or so thinly covered that it is subject to the action of the weather, when rusting will take place causing stains to appear; in some cases expansion due to rust formation results in pushing off particles of the concrete cover. This will obviously spoil both the appearance and the durability of the panels, and it is a serious defect that is difficult to remedy when the work is finished.

Particular care should also be exercised in the preparation of the shuttering and the tamping of the concrete to obtain a good surface when the work is to be left straight from the forms, as otherwise a rough and porous surface will be produced in places; this necessitates patching with cement or cement and sand, which is very unsatisfactory because the patching will disfigure the work and in addition material applied in this manner frequently scales off under the action of frost and the defects are exposed.

Where openings occur in the walling the strength required at the head is usually provided by the addition of some extra reinforcement, and all rods that are intersected by the opening are provided with hooked ends usually passed around the rods that occur at the jambs, head,

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and sill line. Some designers provide diagonal rods across the corners of window openings, these being placed at an angle of 45 degrees with the horizontal and vertical rods and kept in position by the wires. These rods are introduced to prevent cracks which are otherwise liable to occur at these corners when the minimum thickness of concrete is employed.

In some instances the external walls are designed to carry loads brought on by the floor construction, and when this is the case the design will obviously be governed by the circumstances and no fixed rule can be laid down.

It is not unusual to design a portion of a wall on an upper floor as a deep beam equal in depth to the height of the story in which it occurs, and when this is the case extra concrete and steel are provided at the top and bottom with panel filling in between acting as a web between the compression and tension flanges.

In other cases a deep beam is provided by the portion of the wall above or below a window opening and extra horizontal rods are then introduced in the portion which is designed to resist the tension.

The wall filling on the upper floors of a building may therefore be designed to carry considerable loads or it may be merely a screen, but in any case it will obviously assist as a stiffener to the weight-carrying members and it is essential to execute the work with due regard to its importance as part of a structure wherein all the portions must be well tied and knitted together to form one complete unit, as distinct from the older forms of structure where each member was, to a great extent, independent.

Retaining Walls

Retaining walls are built to support or retain earth or water, and there is no more suitable material than reinforced concrete for this purpose. A mass of earth standing alone will not remain with vertical faces, but portions will commence to slide down, particularly when exposed to rain and frost. A simple example of what may be termed the natural tendency of soils is given by pouring sand on to a floor when it will form a mound having sloping sides, but it will be seen that there is a certain angle which the material will maintain even without lateral support. This angle cannot be exceeded unless some support is given, no matter how much the material is piled up or to what extent more material is placed on the top, and this angle is known as the "angle of repose."

The "angle of repose," or the natural angle, will vary with different materials and also in the same material according to the state of consolidation and dryness. Dry sand will usually make an angle of about 30 degrees with the horizontal, whereas clean firm gravel will make an angle of 45 degrees. Material above the natural angle must be supported by artificial means if a vertical face is necessary, and this support is provided by a retaining wall.

The conditions are illustrated in *Fig.* 64, where the natural slope of the soil is indicated by the sloping line *a-b* and the retaining wall is shown in outline above

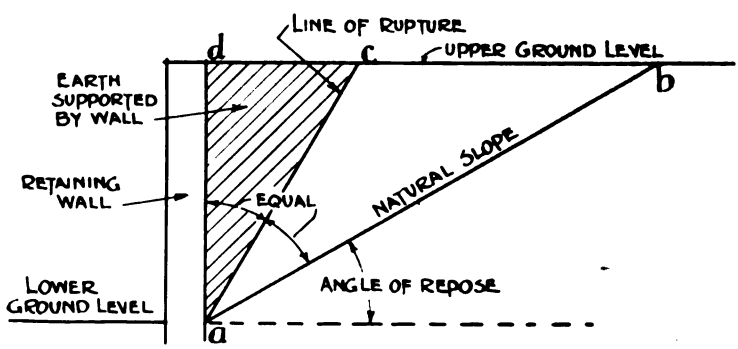

FIG 64

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the lower ground level. Another sloping line *a–c* is marked the "line of rupture," and it is only the wedge-shape portion of earth between this line and the back of the wall which will require to be supported. The removal of the wall would cause this portion only to slip in the first case, because there is a large amount of friction set up by the portion that is tending to slide and the portion which will naturally remain stationary, and to allow for this friction the angle between the natural slope and the back of the wall is divided into two equal parts to give the line of rupture. The earth contained in the triangle between the line *a–c* and the back of the wall shown as *a–d*, presses against the wall and tends to push it over, and the wall must have sufficient weight and strength to resist this tendency. In the calculations it is usual to take a piece equal to 1 ft. run of wall and earth, as this forms a convenient unit, and it will be clear that if a piece 12 in. long is designed and repeated throughout the length of the wall the whole will be satisfactory.

The designer has to consider both the vertical portion of the wall and the base, because the latter must be sufficiently large to anchor the vertical portion and to distribute the pressure over such an area of foundation as necessary to bring the load per square foot within the safe resistance of the soil.

In the case of masonry or brick retaining walls the resistance to the thrust of the earth is dependent on the thickness and weight of the wall itself, and this must be sufficient to prevent overturning. This means that very thick and heavy walls have to be built, involving large expenditure and frequently loss of space, but in the case of reinforced concrete walls the resistance will be given by the weight of the wall *and its strength*, because the material can be designed to offer sufficient resistance to tension at the back of the wall. A brick wall is unable to offer any appreciable resistance to tension, as it is composed of separate units held together by the adhesion of the mortar only, and unless there is sufficient weight on the wall itself acting downward to counteract the outward thrust of the soil, failure will quickly occur. Although it is not proposed to deal with the calculations for retaining walls in detail it may be helpful if some idea of the way in which the pressure acts and how it is determined is explained in order that the reader may realise the force which has to be met by the resistance of the wall.

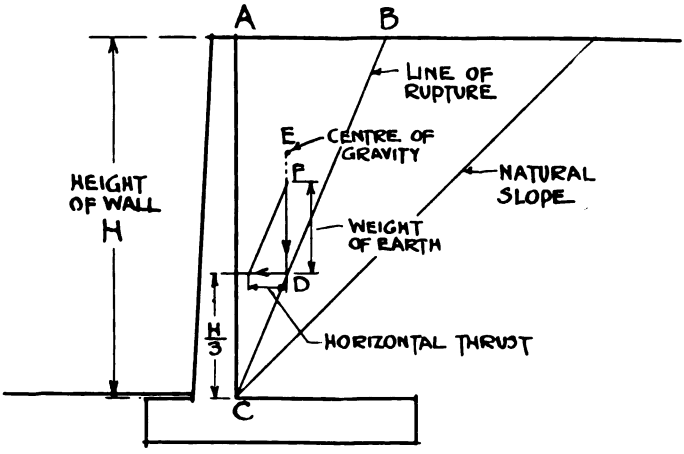

FIG. 65

A diagram of a wall is given in *Fig.* 65 where the natural slope and the line of rupture are indicated. As previously explained, the earth between the line of rupture and the back of the wall is the portion which causes a pressure on the wall and it is the weight of this portion only that need be considered. The

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diagram is set out to scale; the weight of the earth on the triangle A B C is calculated according to the nature of the soil, and this is then considered as acting through its centre of gravity indicated at E. The weight of this earth will act downwards, and a vertical line is therefore drawn downward from the centre of gravity until it cuts the line of rupture as shown at D. As the soil below the line of rupture is theoretically stationary, the weight of the earth above the line which is tending to slide is resisted by the lower soil, and the pressure is caused to act outward against the back of the wall. From the point of intersection at D, therefore, the weight of the earth as calculated is set up to scale from D to F, as shown, and from F a line parallel to the line of rupture is drawn until it intersects a horizontal line drawn from D. The length of this horizontal line then indicates to scale the value of the horizontal thrust on the back of the wall, and it will be found that the level at which this acts is always one-third of H from the bottom, where H equals the height of the wall. It will be seen that the vertical portion of the wall has to act as a cantilever where the base acts as the support, and the load to be carried is represented by a varying pressure the centre of which is situated at one-third the length from the support.

The pressure will cause a bending moment on the wall, the maximum being at the base, and it will gradually diminish towards the top, where it will be nil. This bending moment will cause tension on the back surface of the wall and compression on the front surface in the same manner as explained in connection with beams and slabs. The weight of the wall itself in this case does not add to the stress, but as it acts downward it actually assists in counteracting some of the outward pressure of the soil and thus helps to reduce the bending moment.

It will be realised that in addition to the vertical portion of the wall having sufficient resistance to resist the pressure of the soil as a cantilever it must be sufficiently anchored to the foundation to prevent breaking away and overturning, as indicated in *Fig.* 66, as there is a tendency for it to turn on the front edge, which acts as the fulcrum, and fall forward. This problem can be reduced to leverage and the principles previously explained can be applied without difficulty. The base itself is anchored down by the weight of the earth above, because this would have to be lifted up by the base before the whole wall could overturn.

It will be seen, therefore, that three points have to be considered, (1) the vertical portion of the wall must have sufficient strength to resist the bending

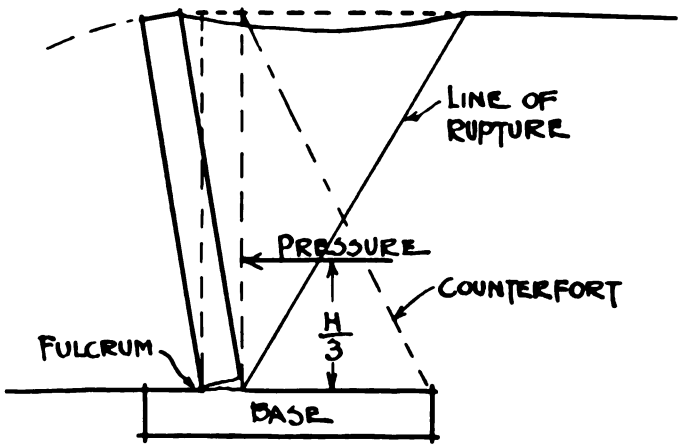

FIG. 66

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moment caused by the outward pressure of the earth, (2) the vertical portion must be securely anchored to the base of the wall to prevent fracture at the junction by leverage, and (3) the base of the wall must be anchored down to the earth either by its own weight, the weight of the earth above, or a combination of the two.

The connection between the base and the vertical portion is sometimes assisted by counterforts or projections at the back of the wall, as shown by the dotted lines in *Fig.* 66, these being placed at intervals and acting as stiffeners to the whole wall.

the necessary resistance; the fulcrum in this instance is at the outer edge of the toe and marked *f* on the diagram.

Under some conditions there is a risk of the whole wall sliding forward on the base without overturning or fracture, and to prevent this the underside of the base is sometimes stepped, as shown by dotted lines, to give vertical surfaces which act as abutments, or a sloping surface may be adopted which is at right angles to the line of thrust, the latter being an inclined line found by a combination of the earth thrust and the downward thrust of the wall itself.

A modified type of wall is shown in

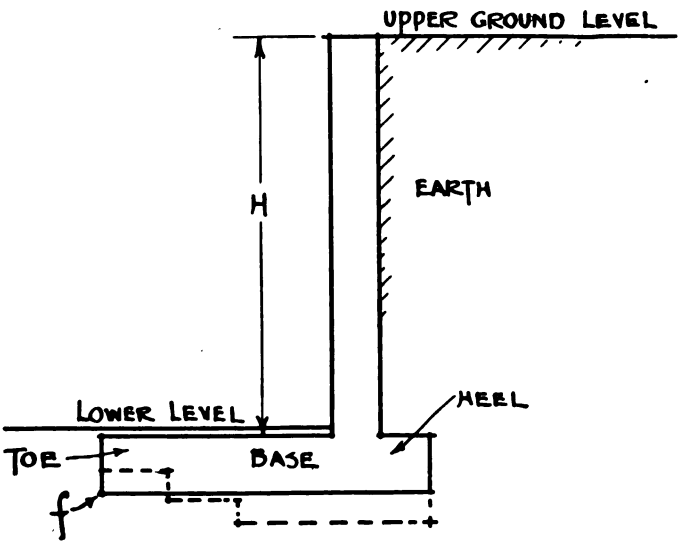

FIG. 67.

Retaining walls are of different types according to circumstances, and it is not always possible to arrange for the base to project at the back of the wall, as, for example, when the adjacent earth at the high level is the property of an adjoining owner who may raise objection to the projection under his land. This is frequently the case, where low-level basements are required, and the type of wall then adopted is one where the base is arranged to project on the front of the vertical portion, as illustrated in *Fig.* 67. In this type of wall there is no weight of earth on the base to prevent overturning, and the base must be designed to give

Fig. 68 which has a projection on both sides, and a special type which is sometimes adopted for the construction of vaults under the pavement is given in *Fig.* 69. In this type the horizontal top portion acts as a cantilever from the vertical wall portion which supports the earth, and the thrust from these two portions is transmitted to the lower soil by means of the projecting cantilever base at the bottom floor level. The total thrust in this case is not only due to the earth pressure at the back of the wall but also to the load of the footpath and pedestrians and the traffic on the road surface immediately above the portion

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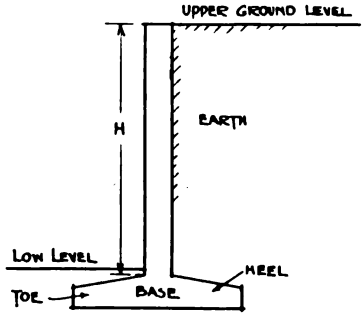

FIG. 68.

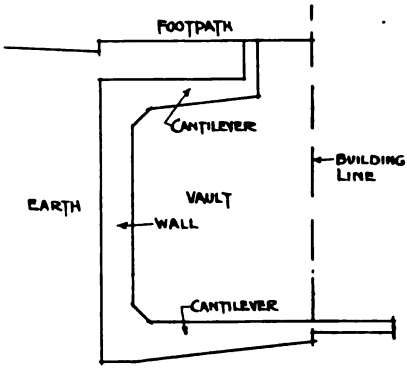

FIG. 69

between the back of the wall and the line of rupture. In some examples the top of the wall is supported by beams which run between the wall and the main framework of the structure ; these beams are then sufficient for the necessary resistance to overturning and the vertical portion of the wall becomes a slab having a span equal to the height of the wall and subject to a varying pressure according to the level under consideration.

These general notes should be sufficient to explain the principles which govern the design, and it will be clear that several points have to be taken into consideration according to the circumstances of each case.

Retaining walls built to resist water pressure follow the same general principles, but matters are somewhat simplified because there is no variation due to weight or angle of repose and the pressure can be accurately determined when the depth of the water is known. A cubic foot of water weighs 62½ lb., and the pressure in lbs. per sq. ft. against the wall will equal the depth of the water in feet multiplied by 62½. The pressure varies directly as the depth, being nil at the top and at a maximum at the bottom, and the centre of the whole pressure will be situated at a level of one-third the height from the bottom.

Fig. 70 shows a section of a wall to retain water when the variation in pressure is indicated by the shaded triangle and the centre of the whole pressure is shown at a distance of $\frac{H}{3}$ from the base which is at the centre of gravity of the triangle. The maximum bending moment at the bottom of the wall will equal the total pressure multiplied by $\frac{H}{3}$. It will be obvious that there is no natural angle or angle of repose for water, and in consequence there is no line of rupture and the whole of the water must be supported ; but the pressure of water varies directly as its depth, and it is

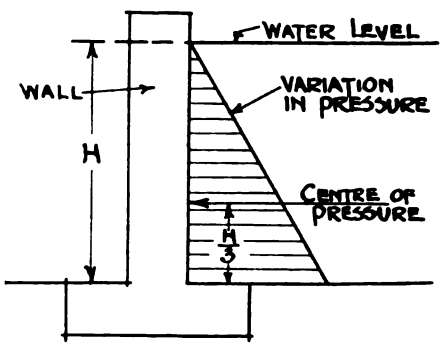

FIG. 70.

not influenced by the shape or size of the tank or other vessel which retains it. In other words, the pressure per sq. ft. on the walls of a tank 10 ft. square by 6 ft. deep is exactly the same as that on the walls of a tank 100 ft. square by 6 ft. deep provided both are filled with water, and the only way to get a reduced pressure is to reduce the depth.

The execution of a reinforced concrete retaining wall calls for the greatest pos-

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sible care and supervision, and it is important that the following points are kept in mind by those in charge of the scheme :—

(1) The formwork must be perfectly rigid and of sufficient strength to resist the pressure of the wet concrete during placing, as the vertical portion will act as a temporary retaining wall for this concrete until the latter has developed sufficient strength to support itself.

(2) No pressure from the soil at the back of the wall must be allowed to come on the concrete until it has developed sufficient strength to act as the retaining wall.

(3) When it is essential to fill in the earth at the back of the wall before the full strength has developed some support must be given to the wall by means of raking shores or struts properly applied to the face of the wall and in accordance with the approval of the designer.

(4) No loads must be placed on the high-level ground close enough to the wall to bring the pressure within the space between the back of the wall and the line of rupture, as this would cause stress on the wall additional to the earth pressure calculated for. It is sometimes necessary to stack materials on the high-level ground for work to be carried out when the retaining wall is finished, and if these are placed close to the wall they constitute a positive danger.

(5) In cases where the permanent high-level soil is finished level with or just below the top of the wall it is important that no soil be temporarily tipped above this level close to the wall, because the weight of this additional soil would be on the portion of earth which is tending to slide and the wall would be called upon to carry this additional load. When the drawings show the earth above the top of the wall with a sloping bank this is a different case, because the designer will have made allowance for the additional pressure and it must not be considered that the same method can be adopted even temporarily with every other retaining wall.

(6) Care must be exercised in any excavation work at the low level close to the toe of the wall after the latter is complete, as the cutting of a deep trench for pipe lines or similar purposes may result in the removal of the necessary earth resistance to sliding forward on the base, and the whole wall may move outward.

(*To be continued.*)

Concrete Memoranda.

In response to several requests from readers we give below the weights of various materials used in reinforced concrete, and propose to make this a regular feature complementary to the Data on the following page. The figures in all cases are average figures, and may vary in different cases.

Cement.—1 cu. ft. of Portland cement is calculated as weighing 90 lbs. ; when filled very loose it may weigh as little as 75 lbs. per cu. ft. ; if packed tight it will weigh 110 lbs. per cu. ft.

Water.—1 cu. ft. of water weighs 62½ lbs. ; 1 gallon = 10 lbs.

Sand and Aggregate.—1 ton = 21 cu. ft. river sand ; 22 cu. ft. pit sand ; 22 cu. ft. ballast ; 23 cu. ft. coarse gravel ; 24 cu. ft. clean shingle ; 40–50 cu. ft. broken brick.

Reinforcement.—Weight of square and round rods per ft. : ¼-in. sq. rods = 0·22 lb. ; ¼-in. dia. round rods = 0·17 lb. ; ½-in. sq. = 0·86 lb. ; ½-in. round = 0·67 lb. ; ¾-in. sq. = 1·92 lb. ; ¾-in. round = 1·51 lb. ; 1-in. sq. = 3·42 lbs. ; 1-in. round = 2·62 lbs. ; 1¼-in. sq. = 5·34 lbs. ; 1¼-in. round = 4·19 lbs. ; 1½-in. sq. = 7·69 lbs. ; 1½-in. round = 6·04 lbs.

Concrete.—Weight of 1 : 2 : 4 concrete per cu. ft. : with coke breeze aggregate, 100 lbs. ; with broken brick aggregate, 125–130 lbs. ; with limestone aggregate, 135 lbs. ; with shingle aggregate, 145 lbs.

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Thames ballast	,,	10	6
Broken brick (¾ in.)	,,	11	6
Best British Portland Cement	per ton 58*s.* to	63	0
"Ferrocrete" Rapid-Hardening Portland Cement	delivered London 10*s.* per ton	extra	
"Super-Cement"	per ton 88*s.* to	93	0
"Lightning" Brand Aluminous Cement	,, 100/- f.o.r. works.		
Ciment Fondu	130/- ex stores		

BOARDING FOR SHUTTERING—		*Sawn.* s.	*Sawn.* d.	*Wrot.* s.	*Wrot.* d.
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—	
3 in. by 4 in.	from £23 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £23 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	0
9/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds	,,	14	6
¼ in. Rounds	,,	15	6

Breeze Slabs per yd. super: 2 in., 1/11; 2½ in., 2/4; 3 in., 2/9; 4 in., 3/6.

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 : 2 : 4—		s.	d.
Do. do. in foundation	per yard cube	48	6
Do. do. in columns	,, ,,	53	6
Do. do. in beams	,, ,,	53	6
Do. do. in floor slabs 4 in. thick	per yard super	5	8
Do do. in floor slabs 6 in. thick	,, ,,	8	5
Do. do. in floor slabs 9 in. thick	,, ,,	12	3
Do. do. in walls 6 in. thick	,, ,,	8	8

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—		s.	d.
From ¼ in. to ⅝ in.	per cwt.	25	0
,, 9/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2⅞ in.	,,	23	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1¼*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2¼*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.* (per bend per cwt.).

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.* (per bend per cwt.).

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	,, ,,	1	1½
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1½
Do. do. as last, in narrow widths	,, ,,	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Carpenters working on old shuttering, 1/10½; Labourers on building works, 1/4½; Men on mixers and hoists, 1/5½; Bar-benders, 1/5½.

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Prospective New Concrete Work.

ABERDEEN.—*Road.*—The Corporation has applied to the M.T. for a loan towards the construction of the extension of Anderson Drive to Holburn Street at the Bridge of Dee.

BINGLEY.—*Houses.*—The U.D.C. has applied for a loan of £15,450 for the erection of 30 houses at Crossflats.

BIRKENHEAD.—*Houses.*—The Corporation proposes to spend £16,343 on the erection of 33 houses on the Derby Park Estate.

BISHOP AUCKLAND.—*Retaining Wall.*—The U.D.C. is considering a proposal for the construction of a retaining wall on the south side of the River Wear, estimated to cost £25,000.

BLYTH.—*Bathing Pool.*—The B.C. has applied for a loan towards the construction of a reinforced concrete bathing pool, estimated to cost £7,000.

BLYTHE.—*Houses.*—The U.D.C. proposes to spend £25,000 on the construction of 50 houses.

BOLTON.—*Sewage Disposal Works.*—The T.C. has applied for sanction to a loan of £132,294 for the construction of sewage disposal works.

BOSTON. — *Reservoir.* — The T.C. has appointed a sub-committee to inquire into a proposal for the construction of a reservoir, which will have a capacity of 33 million gallons.

BRADFORD.—*Road.*—Plans for relief works include a proposal for the construction of a by-pass road at Wyke, estimated to cost £56,000.

BUCKHAVEN.—*Road.*—Direct labour is to be used on the construction of a roadway from Station Road, Buckhaven, to Methil Brae, which is estimated to cost £15,000.

BURNHAM.—*Reservoir.*—The Burnham U.D.C. proposes to construct a 60,000-gallon reservoir.

CHORLEY. — *Houses.* — The T.C. has applied to the M.H. for sanction to a loan of £27,745 for the construction of houses.

CLECKHEATON.—*Houses.*—The Spenborough U.D.C. proposes to erect 50 houses.

DUDLEY.—*Houses.*—The T.C. has applied for sanction to a loan of £25,894 for the construction of 52 houses.

EARSDON.—*Houses.*—The U.D.C. is considering a proposal for the erection of 300 houses on the Park Estate, Shiremoor.

GOLCAR.—*Houses.*—The U.D.C. has applied to the M.H. for sanction to borrow £18,000 for the erection of 34 houses on the Bull Close site.

HENDON (LONDON). — *Houses.* — The L.C.C. is to build 3,000 houses at Hendon.

HINCKLEY.—*Houses.*—The R.D.C. is considering the question of erecting 150 houses in the surrounding parishes.

HOLBEACH.—*Houses.*—The U.D.C. proposes to erect 60 houses.

JARROW.—*Houses.*—The T.C. proposes to erect 46 houses by direct labour, and has applied to the M.H. for sanction to a loan of £20,585.

LEATHERHEAD.—*Sewage Works.*—The U.D.C. has a scheme on hand for the construction of sewage disposal works, estimated to cost £16,000.

MANCHESTER.—*Sewage Disposal Works.*—The Corporation has applied for a loan of £455,000 for the construction of sewage disposal works.

NELSON.—*Houses.*—The General Purposes Committee has recommended the T.C. to apply for sanction to erect 200 houses.

NEWMARKET.—*Houses.*—The R.D.C. proposes to erect 200 cottages during the next two years.

NEWTON ABBOTT. — *Houses.* — The U.D.C. has applied for sanction to purchase 35 acres of land for housing purposes.

NORWICH. — *Concrete Houses.* — The T.C. proposes to erect 100 concrete houses.

NUNEATON.—*Houses.*—The T.C. has applied to the M.H. for permission to erect 200 houses.

POOLE. — *Houses.* — The T.C. has applied to the M.H. for permission to build 100 houses.

SALFORD.—*Houses.*—The Corporation has applied for sanction to borrow £74,740 for the erection of 126 houses in Littleton Road.

SIDMOUTH.—*Sea Defence Works.*—The Sidmouth U.D.C. is to spend £47,000 on sea defence works.

STIRLING.—*Bridge.*—A new bridge is to be constructed in place of the existing Drip Bridge, which connects the counties of Stirlingshire and Perthshire.

STOKE-ON-TRENT. — *Houses.* — The T.C. is considering a plan for the erection of 1,400 houses.

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TAUNTON. — *Concrete Houses.* — The Housing Committee has recommended the Corporation to erect 40 concrete houses.

WALCOT. — *Bridges.* — The Sleaford R.D.C. has applied for a loan of £650 in connection with the construction of three bridges at Walcot, Digby, and South Kyme.

WEALDSTONE. — *Houses.* — The U.D.C. has applied to the M.H. for sanction to borrow £18,580 for the erection of houses.

WITTON GILBERT.—*Road.*—The Durham R.D.C. and the Chester-le-Street R.D.C. are considering the question of laying a road from Witton Gilbert to the main turnpike at Whitehouse Farm.

WORTHING.—*Houses.*—The T.C. proposes to expend £46,524 on the construction of 102 houses at Poor's Ten Acres.

Tenders Accepted.

ABERDEEN.—*Pumping Station.*—The T.C. has accepted the tender of Mr. Wm. Tawse, Aberdeen, for the construction of a reinforced concrete pumping station at the Aberdeen Water Works.

BLACKPOOL.—The Corporation has accepted the tender of Messrs. W. Tarr & Co., Warrington, for the construction of a reinforced concrete bridge and water tank at the public park.

BURNLEY.—*Aqueduct.*—The T.C. has accepted the tender of Messrs. Wellerman Bros., Ltd., Sheffield, at £24,455, for the demolition of the stone culvert over the canal in Yorkshire Street and the reconstruction of a steel and reinforced concrete aqueduct.

DARTON.—*Reservoir.*—The U.D.C. has accepted the tender of Messrs. Swift Bros. & Haslam, Doncaster, at £9,170, for the construction of a covered concrete reservoir in Staincross Lane.

DONCASTER.—*Concrete Factory.*—The Darlington Fencing Co., Ltd., have accepted the tender of Messrs. W. Mullins & Co., Doncaster, for the erection of a concrete factory at Balby Carr.

EXETER.—*Grand Stand.*—The Devon County Athletic Co., Ltd., has accepted the tender of Messrs. W. Dart & Son, Dawlish, at £2,848 7*s.* 6*d.*, for the construction of a reinforced concrete grandstand at the Exeter County Ground.

GLASGOW.—*Concrete Bridge.*—The Corporation has accepted the tender of Messrs. Sir Robert McAlpine & Sons, at £24,630, for the construction of a reinforced concrete bridge over the Duntocher Burn.

LOCHGELLY.—*Water Tanks.*—The T.C. has accepted the tender of Mr. A. Earnshaw, Darlington, for the construction of two reinforced concrete water tanks, of about 200,000 and 90,000 gallons capacity respectively.

MALDON.—*Concrete Bungalow.* — The T.C. has accepted the tender of Mr. G. H. Taylor, Maldon, at £550 10*s.*, for the erection of a concrete bungalow in the recreation ground.

ROMSEY. — *Concrete Bridges.* — The R.D.C. has accepted the tender of Mr. A. E. Farr, Victoria Street, S.W.1, at £2,800, for the reconstruction of three bridges in reinforced concrete.

SEDGEFIELD. — *Storage Tank.* — The R.D.C. has accepted the tender of Mr. J. W. White, Sunderland, at £1,079, for the construction of a reinforced concrete storage tank at the sewage pumping station.

SELKIRK.—*Weir.*—The Joint Dam Committee has accepted the tender of Messrs. Wm. Taylor & Son, Glasgow, at £10,590 4*s.* 5*d.*, for the construction of a concrete weir across the River Ettrick.

STAINTON-IN-CLEVELAND. — *Culvert.* —The Middlesbrough R.D.C. has accepted the tender of Messrs. J. Pearson, Ltd., Stainton-in-Cleveland, for the construction of a reinforced concrete culvert at Maltby.

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EDITORIAL NOTES.

Concrete Design.

To discussions amongst architects on architectural styles there is no end, and sometimes reinforced concrete is more or less casually referred to. True it is more often than not mentioned at the end of the lecture and side-tracked with a remark or two to the effect that the material is something quite new, that its artistic progress has not been all one could wish for, and that only the future can tell what manner of flower will blossom forth from this most unpromising seedling—and it is generally to be inferred that it is unpromising because it is unwelcome. But the fact remains that concrete as a material to which their art might be applied is being talked about by architects, and in some cases thought about—a very different thing.

The question is again raised in a new book * by Mr. Martin Shaw Briggs, who delves below the surface of things to get at essentials and find the reasons of

* *A Short History of the Building Crafts.* By Martin S. Briggs, F.R.I.B.A. (Oxford: Clarendon Press. Price 8*s.* 6*d.* net.)

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practical construction on which all good architecture must necessarily be based. In this volume he deals with all the principal building crafts and traces the development throughout the ages of the methods we know to-day. Concrete is given a chapter to itself, and this we commend to those architects who are apt to look askance on the modern use of the material on the grounds that it is new and has no tradition from the remote past. "The builders in the great periods of history," says Mr. Briggs, "welcomed novel materials and methods as they were invented," and therein lay the secret of their success. If the material were sound, an honest endeavour was made to make the best use of it; there was no question as to whether the form and expression which best suited it followed precedent; where no tradition existed, the beginnings of a tradition were made. "The ultimate future of design in reinforced concrete," the author continues, "is full of possibilities, and calls for all the genius of the younger generation. It is certain we shall never solve the problem by a slavish following of the past, for it is a problem that has never been faced before."

One of the chief characteristics of concrete is its adaptability. If precedent in design only were in question there would be no architectural misgivings at all, for any style can be followed in its minutest detail. The problem is to find a style for a new material, and in the attempt a more open mind might well be brought to bear in academic discussion. We are told, for instance, that the colour and texture of concrete from the forms are hopeless, but cannot the same be said of marble and stone before they are polished and tooled? Methods of treating the surface of concrete so that the material of which it is made is exposed are producing excellent results, and the process is quite analogous to the surface treatment of marble and stone. It is frequently stated that the slenderness of reinforced concrete members when designed with the utmost economy of materials makes a building appear to lack strength and stability. But is it essential that they should be so thin? No architect designs a stone column of a minimum diameter to carry the weight above it: columns are designed to be in harmony with the proportions of the remainder of the building, and frequently contain twice as much material as would be necessary to meet the bare requirements of structural stability. This being so, why cannot the same practice be applied to reinforced concrete construction? It will generally be found that the buildings which architects call "spidery" have been built from the utilitarian point of view only, and that architects have had no hand in them. If ever the "concrete style" is to be evolved on satisfactory lines an open mind must be kept on these and other points. While concrete is looked upon as an unwelcome interloper no progress will be made; it must be accepted as the builders of the past accepted new ideas and materials. Certain it is that concrete cannot be denied its position as the chief building material of the future.

We commend Mr. Martin Shaw Briggs's book to all interested in building construction. It forcibly brings out the attitude of past masters towards their craft, and its study should do much to help present-day architects to look upon their work in the same light.

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Progress of Concrete Roads.

WITH the road problem so much to the fore just now it is natural that those interested in different types of construction should be advocating the merits of their respective materials. Apart from the legitimate "pushing" of their own methods, however, we find that certain trade interests are decrying concrete roads. It is even being suggested that the progress of concrete road construction in America is declining, and that this type of road is discredited by American engineers. In view of these statements, the following official figures are of particular interest. From January 1 to June 6, 1925, contracts were placed in the United States for the construction of no less than 51,080,153 square yards of all-concrete roads. This figure compares with 42 million square yards in the first half of 1924, and 33 million square yards in the first half of 1923. These official figures are sufficient answer to any suggestion that the concrete road is declining in popularity in the United States, and although the total mileage so far laid in this country is, of course, not comparable with the American total, the figures in our possession show that the mileage of concrete roads in Great Britain is increasing quite as fast as across the Atlantic.

An Elementary Guide to Reinforced Concrete.

MR. ALBERT LAKEMAN'S series of articles which has appeared in this journal under the title "The Foreman's Guide to Concrete" have now been reprinted in book form, under the title "Elementary Guide to Reinforced Concrete." This is in response to the large number of appreciations we have received, and in order that it may be within reach of those for whom the articles were primarily intended the book is published at the low price of 2*s*. It can safely be said that no other book sets out so simply the principles of reinforced concrete design and construction. It has been specially written for the beginner by an author particularly well fitted for the task, and is written in such simple language that it is readily understandable by those with no previous knowledge of the subject whatever and without reference to any other work. A feature of the volume is the large number of clearly-drawn diagrams which illustrate the explanations of terms in use and designing principles, and the analogies with everyday things which help the reader to realise at once the points the author wishes to emphasise. For the architect or student desirous of gaining a knowledge of the theory and practice of reinforced concrete with a minimum of labour there is no better medium. It is also hoped that the book may find its way into the hands of clerks-of-works and builders' foremen, as it is certain that the knowledge which it conveys in such a pleasant manner cannot fail to be conducive to better workmanship. A realisation of the reason for placing reinforcement in the exact position indicated in the drawings, and of the danger that the whole design may be ruined and an unstable building result if the drawings are not carefully followed, will not only make for better building, but will also give the foreman an added interest in his work. The book is now ready, and may be obtained from Concrete Publications, Limited, 20, Dartmouth Street, Westminster, S.W.1.

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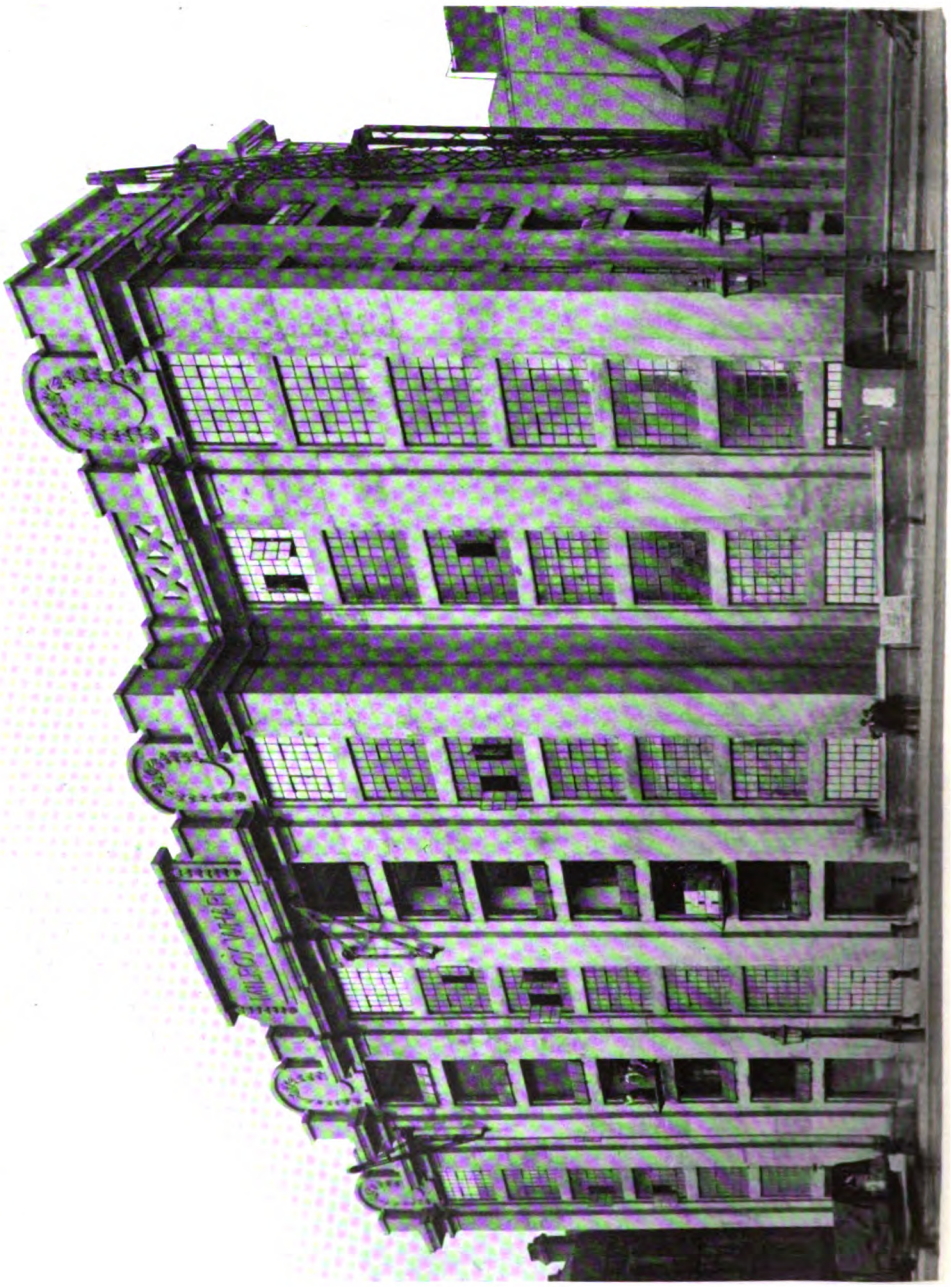

Milroy Wharf, Blackfriars. [Mr. A. Howell Ridge, Architect.

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Milroy Wharf, Blackfriars.

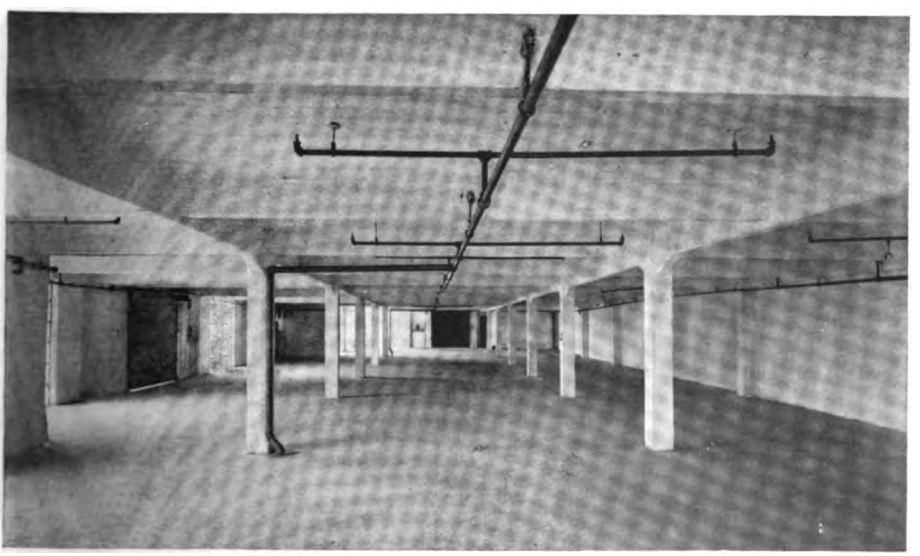
We give some illustrations this month of another important reinforced concrete building recently completed in Central London, known as Milroy Wharf, situated on the waterside at the southern end of Blackfriars Bridge. It is a complete reinforced concrete building, with an untreated concrete surface, and as will be seen from the photographs careful attention to proportion and design has resulted in a dignified and pleasing structure without recourse to expensive trimmings or stone covering. It is interesting to note that the first building with any architectural pretentions to be built on the Surrey side of the river since the erection of the new County Hall at Westminster is of reinforced concrete. Certainly there is plenty of scope here to add to the amenities of London by replacing the conglomeration of dust-shoots and wharfs which disfigure this side of the water and make the view from the Embankment an eyesore, and it is gratifying to see that concrete is leading the way in one of the most badly-needed improvements in London.

The building is for a firm of wharfingers, and is used as a warehouse for the storage of provisions. It comprises two basements, ground floor, and six floors above, with a frontage to Blackfriars Bridge Approach of 146 ft. 6 in., a frontage to the river of 38 ft., and to Upper Ground Street of 43 ft. 9 in. The cubical extent of the building is about 560,000 cubic feet, and the storage capacity is about 13,000 shipping tons.

The foundations were formed of about 200 reinforced concrete piles of an average length of 14 ft. The interior columns are octagonal, with a diameter of 2 ft. 8 in. in the basement diminishing to 1 ft. 9 in. on the sixth floor. The main beams are 18 in. by 12 in. and the secondary beams 14 in. by 9 in. The reinforced concrete floors are 5 in. thick, each floor covering an area of about 6,300 superficial feet.

The river front is carried on cantilevers so as to take all the weight from the old river wall.

The architect is Mr. A. Howell Ridge, of Newgate Street, E.C.1, and all the reinforced concrete is on the Hennebique system, from the design of Messrs. L. G. Mouchel & Partners, Ltd. The contractors were Messrs. Gray's Ferro-Concrete Co., of Glasgow.



[*Mr. A. Howell Ridge, Architect.*

Milroy Wharf, Blackfriars.

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[*Mr. A. Howell Ridge, Architect.*

Milroy Wharf, Blackfriars. (*See p.* 281.)

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Staithes Harbour Improvement.

By James Mitchell, M.Inst.C.E., M.I.Mech.E.

THE quaint old fishing-village of Staithes lies on the coast of Yorkshire, about 10 miles north of Whitby, and is situated in a deep ravine through which a small stream enters the sea. It used to be a lifeboat station, but the lifeboat was removed a few years ago owing to a re-arrangement of the life-saving arrangements consequent on the introduction of motor lifeboats. The lifeboat house and the slipway still remain, however, both in excellent preservation, and were of great service during the progress of the new works, the former being used as a cement and general store while the latter carried a railway track for the transport of sand, gravel, and cement across the harbour, and for the removal of boulders, broken rock, etc., from the excavations.

The old Fish Quay was very much exposed to the sea, and as the result of representations made to the Board of Agriculture and Fisheries, and especially in view of the widespread unemployment due to the closing down of one of the large ironstone mines in the neighbourhood, a free grant of money was made for the purpose of carrying out the scheme of improvement hereinafter described. This scheme consisted, broadly, of the construction in mass-concrete of two breakwaters (each 400 ft. long) and an intercepting wall (100 ft. long), the blasting of a boat channel, and the removal of a shoal of boulders which lay just outside the new harbour entrance.

The breakwaters and the intercepting wall are founded throughout upon hard shale-rock, the layers of which dip towards the south-east at an angle of about 5 degrees. The harbour entrance faces about north-east. It was originally intended to construct the east and west breakwaters simultaneously, but owing to the difficulties imposed by the very limited amount of storage room available for the concrete materials this idea was given up, the West Breakwater being completed before the east one was begun.

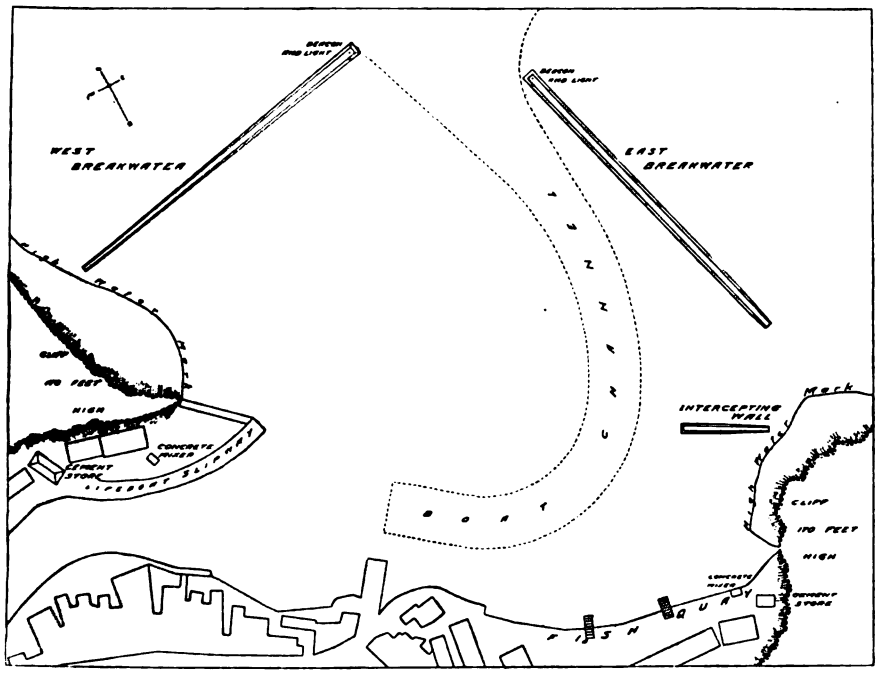

Fig. 1.—General Plan of Harbour.

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At the site of the breakwaters the sea-cliffs are about 170 ft. high and nearly perpendicular, as may be seen in *Figs.* 4 and 5, which show the East Breakwater under construction with the completed West Breakwater in the background. The only practicable road into and through the village is narrow and tortuous, while a portion of it, Staithes Bank, has a gradient of about 1 in 5. The road on the other side of the stream is so steep as to be quite impracticable for heavy traffic, and as permission could not be obtained for laying a tramway track across the only available footbridge the problem of getting plant and materials to the site, and especially to the West Breakwater, was a very difficult one. It was made more so by the fact that the local authority would not permit heavier loads than five tons—including the weight of the vehicle—to be carried on the road through the village.

The idea of lowering the sand, gravel, and cement, or the finished concrete, from the top of the cliff at the root of one or both of the breakwaters was carefully considered. It was, however, finally decided to bring all the concrete materials to the Fish Quay on the east side of the harbour and to transport those required for the West Breakwater across the harbour by means of a 2-ft.-gauge railway laid on the sea bottom. This plan had the drawback that it could only be operated during low tide, and had to be suspended during floods in the stream and when the storage accommodation at the west side was fully occupied, while the railway was liable to damage from rough seas and from the damming of materials brought down by the stream during floods. It worked on the whole, however, very satisfactorily.

The sand and gravel for concrete-making were brought from a sea beach about 5 miles distant, and were screened at the site of the works, the screening helping to provide occupation for the workmen during periods when the sea was too rough for depositing concrete. The material was taken from the beach by horse and cart, and dumped at a point above high-water mark where it could be picked up by a 4-ton motor-wagon. The motor-wagon conveyed it to the top of Staithes Bank, where it was again dumped. From thence it was taken by cart to the Fish Quay where it was stored, that required for the West Breakwater being transported across the harbour when required, as already described.

The supply was a somewhat precarious one, owing to the fact that the beach was in general extremely sandy, the gravel being got only in small patches and after rough seas. The storage room available at the top of Staithes Bank, on the Fish Quay, and at the west side of the harbour, was in each case extremely limited. When to these difficulties are added the interference by rough sea with the concreting operations, and the troubles with the transport across the harbour, it will be seen that the task of keeping the concrete-mixer constantly supplied was a somewhat anxious one.

The over-size material was screened from the gravel by a $1\frac{1}{2}$-in. square mesh, and the sand by a $\frac{3}{16}$-in. square mesh. The concrete was mixed by a $\frac{1}{2}$-yd. petrol-driven mixer, the mixture being $1 : 2 : 3\frac{1}{4}$ where no displacers were used and $1 : 2 : 3$ where displacers were used. The sand and gravel were measured separately, and were conveyed to the mixer in $\frac{1}{2}$-yd. tipping wagons, the quantities in a batch being arranged to suit one sackful of cement ($\frac{1}{12}$ton) as it was delivered from the maker. The measuring-wagons had narrow strips of iron riveted to them to indicate the level of the surface of the sand (which was put in first), and of the gravel (which was put on top of the sand). The use of displacers was optional, but owing to the hindrance of the concreting operations caused by their use, and to the restrictions imposed by the tides, they were only used in a very limited portion of the landward end of the East Breakwater.

For the conveyance of the concrete from the mixer to the breakwater a 2-ft.-gauge railway was laid in each case. *Fig.* 3 shows the track for the East Breakwater. This was carried on 12 in. by 6 in. pitch-pine timbers supported on concrete piers, the spans being about 20 ft. As the railway was extended along the breakwater wooden plugs were embedded in the concrete, by means of which the sleepers were spiked down in order to prevent the sea from sweeping the track away.

The forms for the wall-faces were made in panels of 9 in. by $2\frac{1}{2}$ in. redwood plank-

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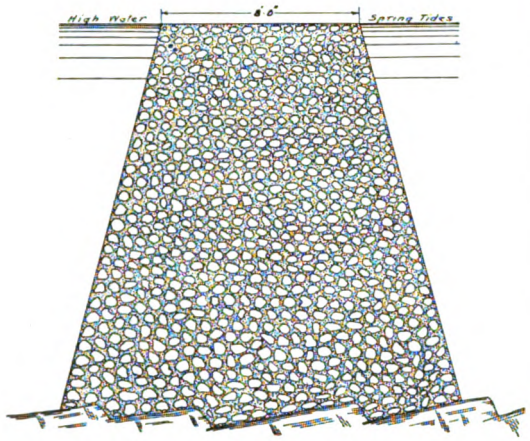

FIG. 2. TYPICAL CROSS SECTION OF BREAKWATERS.

ing, planed on one side and both edges, and thicknessed, the backings being of 9 in. by 3 in. planks. The panels were 20 ft. long and 5 or 6 planks wide. The breakwaters and the intercepting wall have a uniform batter on each side of 1 in $3\frac{3}{4}$, but as the top-width gradually increased as they extended seaward the stuntheads were built up of separate planks instead of being made in panels. A 9 in. by 3 in. upright was bolted to each of the side panels close to the end, and to this the stunthead planks were spiked. After a casing had been filled with concrete, and when the next length of shuttering came to be set, these uprights were unbolted and removed, thus allowing the stunthead planks to be taken away. The new panels of shuttering were then butted against the old ones and the joint covered on the outside by a 9 in. by 3 in. upright, to which both panels were spiked. Through tie-rods of $\frac{5}{8}$-in. diameter were used—generally six in each length of casing—and owing to the large amount of batter they were fitted with large bevelled hardwood washers, in addition to the ordinary square iron ones. *Figs.* 4 and 5 give a good general idea of the shuttering when in position.

Owing to the varying top-widths and the batter of the walls, and to the sloping and irregular surface of the rock, the setting out of the bottom casings was somewhat troublesome. As these casings were in general erected and fixed during one low tide and filled with concrete during the following low tide they were very liable while in their empty condition to be displaced by rough seas, owing to their buoyancy and to the slippery nature of the rock-surface. It was at first attempted to hold the casings down by wire ties attached to hookbolts cemented into the rock, the ties being tightened by twisters. This not proving satisfactory, the casings were attached by means of coach-screws to eyebolts let into the rock. Finally a few tons of pig-iron were purchased for use as loading, the pigs being laid upon transverse planks spiked to the top edges of the casing. The pigs were easily and quickly put in place and removed, and proved quite satisfactory in use.

A considerable quantity of boulders and some loose rock had to be removed from the sites of the breakwaters and the intercepting wall, and where the rock surface was free from boulders it was mostly grown over with seaweed which

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Fig. 3.—East Breakwater (partly completed) and Temporary Approach-Railway.

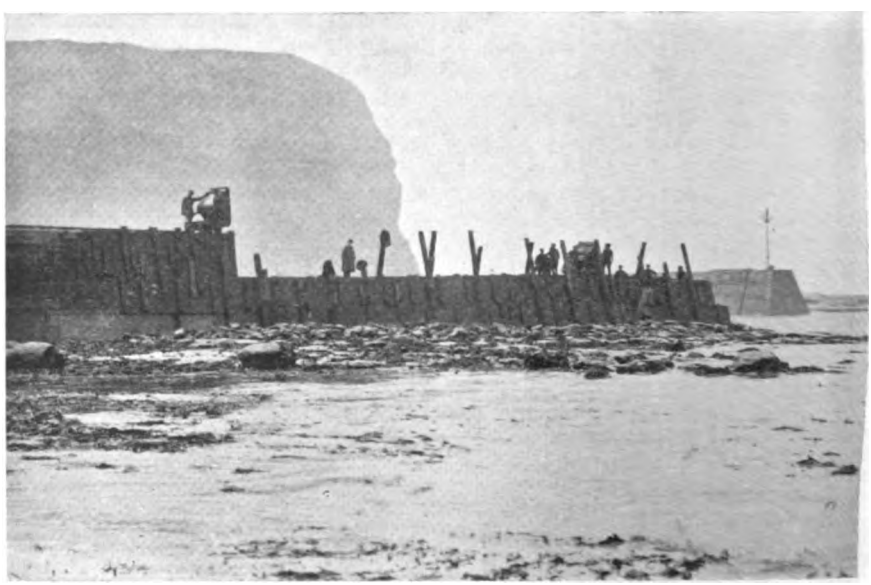

Fig. 4.—East Breakwater during Construction, showing Concrete Casing in Position.

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had to be removed by scraping. In setting the lower casing-panels no particular attention was paid to keeping them level, provided the inclination was not too great. The spaces between the bottom edges of the casing and the rock surface were dealt with by tacking to the foot of the inside of the casing an apron of strong jute-tarpaulin, of a width sufficient to extend inwards about 9 in. on the surface of the rock. When the concrete was deposited on this apron it sealed the bottom edges of the casing against the wash of the sea. Where the space was more than about 5 in. deep the apron was supported against the bursting effect of the soft concrete, in some cases by planks roughly fitted to the rock surface, and in other cases by bars of pig-iron laid against it. The butt-joints between the various panels, and the corners where the stunthead joined the side panels, were covered internally with strips of tarpaulin. The through tie-rods of the lowest casings were made sufficiently long to be utilised for the fixing of 9 in. by 3 in. upright "soldiers" (see *Figs.* 4 and 5) to which the panels of the upper casings were attached. The panels of shuttering were generally conveyed to the site by floating them out during the ebb-tide.

The concrete was conveyed from the mixer in end-tipping wagons each carrying 20 cu. ft.—two batches. In the case of the top casings it was dropped from the wagon direct into the casing. In the case of the lower casings it was tipped into a short shoot, or on to an inclined plane made of ¾-in. boarding.

As the greater portion of the breakwater sites did not bare at low-water of neap-tides, it was necessary to push out as much of the lowest portion of the walls as possible during spring-tides, so as to enable concreting to be carried on "in the dry" during the neap-tides. As this lowest portion advanced a 2-ft.-gauge railway carrying an end-tipping wagon was laid on it, and into this wagon concrete was tipped from the wagons running on the top of the breakwater (see *Figs.* 4 and 5).

During deposition the concrete in the casings was handled by men wearing rubber thigh-boots, and was of such a consistency that the men sank to the knees in it. After the placing of the concrete had been completed and the surface levelled it was covered by sheets of jute-tarpaulin, on which were laid pieces of old steel-plating ½-in. thick and of a size convenient for handling—about 3 ft. by 1½ ft.

The protection of the finished top-surfaces of the walls until they hardened was somewhat difficult, as they were situated at the level of high water of ordinary spring-tides, and it was made more so by their varying widths. During neap-tides and with a smooth sea they could be left uncovered, and this produced the best results. When protection was necessary—which was mostly the case—they were covered with tarpaulin sheets on which were laid panels of ¾-in. boarding, nailed at the edges to the concrete casing.

The concrete was almost entirely deposited "in the dry." In the few cases where it was necessary to deposit it amongst water a bottomless box, measuring 5 ft. by 3 ft. in plan and 2½ ft. deep, and having lifting handles attached to it, was used. The box was set on the ground and concrete tipped into it till it was full. It was then lifted slightly, to allow the concrete to escape round the bottom edges, and tipping continued until an "island" of concrete was formed. The box was then removed, and as tipping continued the island increased in area and the water within the casing was displaced with a minimum amount of damage to the concrete.

A considerable number of casings were carried away by the sea and a few panels broken. Fortunately, however, owing to the set of the local currents it was possible to salve almost the whole of the timber. Owing to the excellent proportions of the concrete it rapidly attained a considerable amount of strength, and thus very little of it was destroyed by the sea. In general, even when there was a considerable swell in the sea, it could be confidently reckoned that if the concrete could be got in and covered and loaded with steel plates before the sea came on to it there was little danger of its being damaged.

Some trouble was caused by a large hollow in the rock, near the outer end of the East Breakwater, which was filled with heavy boulders, compacted with gravel, mud, and sea growths. The excavation of this material was carried on within small dams, formed with clay

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Fig. 5.—East Breakwater during Construction, showing Advance of Lowest Section of Wall.

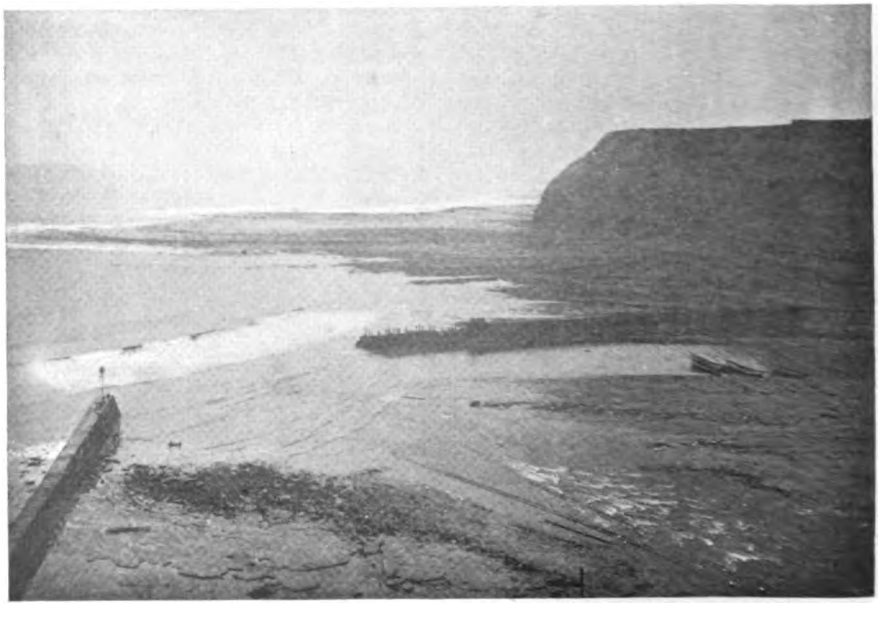

Fig. 6.—General View of Breakwater at Low Water; East Breakwater partly completed.

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in bags, the water being removed by means of trench pumps.

Each breakwater is equipped with the usual ladders and bollards and has at its outer end a mast carrying a beacon and a dioptric-lens lamp, the lamp having a special opening at the back by means of which, in addition to the light shining seawards, a small beam is shown towards the harbour for the guidance of the fishermen when setting out for sea. During the progress of construction a temporary coloured-light was exhibited at night on the seaward end of the work.

At the entrance to the new harbour a dangerous shoal of boulders, ranging in weight up to about three tons, was removed to the level of low-water of ordinary spring-tides. It was at first proposed to remove them by means of a hand derrick-crane erected on the outer end of the West Breakwater, the boulders being brought within range of the crane by tipping wagons running on a railway laid on the sea bottom, and being removed from the crane by a similar arrangement on the top of the breakwater. Owing, however, to the heavy seas to which the crane would be subjected in this position its site was changed to the sea bottom, just within the outer end of the breakwater. Even here it was twice overthrown by the sea, and the idea of using it was then abandoned. Ultimately the boulders, after they had been broken to suitable sizes by hammers and wedges, were removed by men with hand barrows. Some of them were conveyed directly to the seaward side of the East Breakwater and there deposited, but the greater number were loaded on to tipping wagons and conveyed by horses to the foot of the cliff on the harbour side of the West Breakwater. A portion of the railway used for the purpose is to be seen in *Fig.* 6. As the shoal was at all times partly at least submerged, the men worked in rubber thigh-boots, being much impeded by the irregular and slippery nature of the ground. The larger boulders were at first broken up by blasting, but owing to the short time available for work in a tide the stoppage of the men thereby involved rendered it cheaper to use hammers and wedges for the purpose, especially as the nature of the stone lent itself to this method of disposal. Owing to the fact that the work of removal could only be carried out during the lowest of spring-tide ebbs, and that it was interfered with to a considerable extent by rough seas, floods in the stream, and bad weather, it was very tedious.

For the excavation of the boat channel, which was, of course, done entirely by tidework, the rock was bored by twist drills worked by hand ratchets, and was blasted with black powder made up in the form of cartridges and fired by ordinary miner's-squibs. After the larger pieces had been further reduced by hammers and wedges it was loaded into tipping wagons and conveyed by horses to the foot of the cliff on the harbour side of the West Breakwater, the railway laid for the transport of the concrete materials across the harbour being utilised for the purpose. Portable acetylene-lamps were used throughout for carrying on work during the night.

The works were designed by Messrs. J. Watt Sandeman & Son, civil engineers, of Newcastle-on-Tyne, and constructed under their supervision, Mr. A. J. Clark, B.Sc., A.M.Inst.C.E., being Resident Engineer. The consulting engineer for the Board of Agriculture and Fisheries was Mr. N. G. Gedye, O.B.E., M.Inst.C.E. The contractors were Messrs. Brims & Co., Ltd., of Newcastle-on-Tyne and Westminster, for whom the writer acted as agent.

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Effect of Oils and other Liquids on Concrete.

THE following notes on the effect on concrete of oils and miscellaneous liquids, with hints on the method of protective treatment where required, have been approved by the American Society for Testing Materials. Notes (*a*) give the effect of the various liquids on untreated concrete, and notes (*b*) the surface protective treatment advocated :—

MINERAL OILS 30 DEG. BEAUMÉ OR HEAVIER : (*a*) Good concrete unaffected ; very slight surface penetration. (*b*) Good concrete well spaded or cement mortar finish sufficient.

FUEL OILS ABOVE 30 DEG. BEAUMÉ, DISTILLATES, GAS AND LUBRICATING OILS : (*a*) Good concrete unaffected ; more penetration than for heavy oils. (*b*) Coatings of the magnesium fluosilicate class required for storage tanks.

KEROSENE, GASOLINE, BENZINE : (*a*) Good concrete unaffected ; considerable penetration. (*b*) Gasoline-proof coatings producing glazed surface, or treatment with iron compounds.

ANIMAL OILS (SOLID FATS) : (*a*) May attack concrete slowly, particularly if in melted condition. (*b*) Oil-resisting compounds.

ANIMAL OILS (LIQUID FATS—including neat's-foot oil and menhaden oil) : (*a*) No effect on good concrete.

ANIMAL OILS (LIQUID FATS—including cod-liver oil, whale oil, and seal oil) : (*a*) More or less disintegrating. (*b*) Oil-resisting compounds.

COCOANUT OIL : (*a*) Some action if stored in close tanks ; progressive disintegration if in contact with surfaces exposed to air. (*b*) Sodium silicate or magnesium fluosilicate treatment sufficient for closed tanks ; proprietary compounds have been effective on floors.

LINSEED OIL, TURPENTINE, ROSIN OIL : (*a*) No effect on good concrete ; considerable penetration of turpentine.

COTTONSEED OIL : (*a*) No action if stored in closed tank of good concrete.

RAPE SEED, CASTOR, AND MUSTARD OILS : (*a*) Progressive disintegration if in contact with surfaces exposed to air.

OLIVE OIL : (*a*) Some action. (*b*) Coatings of the varnish type effective.

TANNING LIQUORS : (*a*) Acid liquors have considerable effect ; other tanning extracts have no action. (*b*) Bituminous acid-proof paints are effective for tanks holding acid tanning solutions ; good concrete with or without surface mortar finish is sufficient for other tanning extracts.

ACETIC ACID (VINEGAR, ETC.). (*a*) Attacks concrete. (*b*) Bituminous acid-proof paints are effective ; paraffin coatings applied hot are also useful for tanks.

MOLASSES : (*a*) No action if stored in closed concrete tanks.

SULPHURIC ACID SOLUTIONS : (*a*) Progressive disintegration, particularly if concrete is subject to abrasion. (*b*) Bituminous acid-proof paints or mastic coatings are effective.

Effect of Lignite Coal on Concrete.

IT was found that the sand used on some large concrete works by the Engineering Department of the City of Edmonton contained a percentage of fine lignite coal, and an investigation of the effect of this foreign substance on the strength of concrete has been made by Messrs. I. F. Morrison and H. R. Webb, of the University of Alberta. It was found that the sand in question gave an increased strength of 4 per cent. when washed free from the coal.

The conclusions drawn from the investigations are as follows :—

(1) Small amounts of finely-divided lignite coal lower the strength of concrete up to periods of one year. The amount of coal for which this effect was observed was only 0·05 per cent. by weight of sand.

(2) The effect of amounts of this coal greater than 0·05 per cent. is not appreciably more than that for 0·05 per cent., up to amounts of 1·0 per cent. by weight of sand.

(3) The decrease in strength is about 8 per cent. for 0·05 per cent. of coal and is not comparable with the decrease in strength due to the presence of organic acid as indicated by the colorimetric test.

(4) The colorimetric test cannot be taken as a criterion of inferior quality for sands containing small amounts of lignite coal.

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The Deflection of and the Unit Influence Lines of Deflection for Girder Beams.

(*a*) Simple, (*b*) Encastre and (*c*) Continuous: and for which the Product *EI* is Constant.

By H. W. Coultas, M.Sc.(B'ham.); B.Sc.(Leeds),

Introductory.

THE left-hand support of the beam, whether simple or continuous, is taken as the origin: all moments acting in a clockwise direction to the left of a section are taken as positive, and all moments acting in an anti-clockwise direction to the left of a section are taken as negative. The horizontal direction of left to right from the origin is taken as positive: a downward deflection of the beam below its horizontal position is positive; and an upward deflection above its horizontal position as negative.

Using the sign of moment taken in the fundamental equation

$$EI\frac{d^2y}{dx^2} = M$$

the correct sign of deflection is always obtained.

The method adopted of finding the deflection of a beam is that given by Mr. W. H. Macaulay in the *Messenger of Mathematics* (No. 573, January, 1919, xlviii) for any irregular system of loading.

Let a simple beam be loaded as in *Fig.* 1.

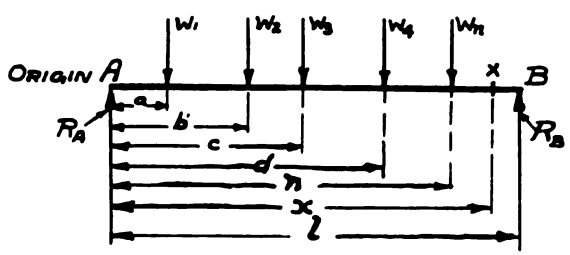

Fig. 1.

R_A and R_B can be found by general methods.

Take any section X (between the last load W_n and the support B) and distant x from the origin.

The moment at

$$X = M_x = EI\frac{d^2y}{dx^2} = -R_Ax + W_1(x-a) + W_2(x-b) + \ . \ . \ + W_n(x-n)$$

Integrating twice,

$$EIy = -R_A\frac{x^3}{6} + \frac{W_1}{6}(x-a)^3 + \frac{W_2}{6}(x-b)^3 + \ . \ . \ . \ \frac{W_n}{6}(x-n)^3 + Px + Q.$$

When $x = 0$, $y = 0$, then $Q = 0$, because all terms such as $(x - a)$ which become negative for a particular value of x, are neglected.*

* The proof of this is given in the article "The Elastic Equations for Beams," by W. D. Womersley, M.A., B.Sc., *Concrete and Constructional Engineering*, July, 1925.

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When $x = l$, $y = 0$, then

$$0 = R_A\frac{l^3}{6} + \frac{W_1}{6}(l - a)^3 + \frac{W_2}{6}(l - b)^3 + \ldots Pl,$$

an equation from which P can be found.

This method of finding the constants P and Q also holds good for the continous beam. Q will be equal to zero, whilst P can be found by giving x the value of the length of the first span and using the corresponding value of y at the end of the first span. If the supports are all at the same level, then this value of y will also be zero.

If on the beam there is also a uniformly distributed load and it does not extend to the section X taken, it is imagined that the load does extend to the section X with another small length of load acting upwards to neutralise the imaginary portion acting downwards.

Deflection of a simple beam AB of length l carrying a unit dead load at C a general distance nl from the origin (Fig. 2).

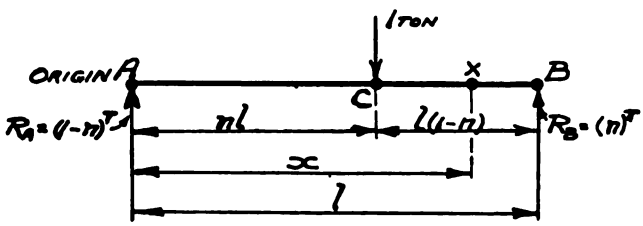

Fig. 2.

$$R_A = (1 - n)^T : R_B = (n)^T.$$

Take any section X between C and B.

$$\text{Then } EI\frac{d^2y}{dx^2} = M_x = -R_Ax + (x - nl)$$

$$= -(1 - n)x + (x - nl).$$

$$EIy = -(1 - n)\frac{x^3}{6} + \frac{(x - nl)^3}{6} + Px + Q$$

when $x = 0$, $y = 0$, then $Q = 0$.
when $x = l$, $y = 0$.

$$\therefore\ 0 = -(1 - n)\frac{l^3}{6} + \frac{l^3}{6}(1 - n)^3 + Pl.$$

From which $P = \frac{l^2}{6}(2n - 3n^2 + n^3)$.

For values of x between 0 and nl

$$EIy = -(1 - n)\frac{x^3}{6} + \frac{l^2}{6}(2n - 3n^2 + n^3)x \quad \ldots \quad (1)$$

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For values of x between nl and l

$$EIy = -(1-n)\frac{x^3}{6} + \frac{(x-nl)^3}{6} + \frac{l^2}{6}(2n - 3n^2 + n^3)x. \quad . \quad . \quad (2)$$

Using either equations (1) or (2) for

$$x = \frac{l}{2} \text{ when } n = \frac{1}{2}$$

$$EIy = \frac{l^3}{48}$$

$$\text{i.e. } y = \frac{l^3}{48EI} \text{ per unit load.}$$

If the load was W tons then

$$y = \frac{Wl^3}{48EI}.$$

To find the deflection at a constant section Z distant z from the origin as a unit load crosses the beam. This is, to obtain the Unit Influence Line of Deflection for the Section Z (Fig. 3).

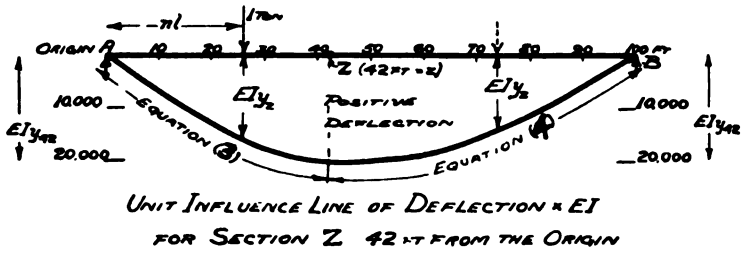

Fig. 3.

(*a*) Let the load be between A and Z and at a distance of nl from A. The deflection at Z for any value of nl less than z will be found by adjusting equation (2) by substituting z for x. Then

$$EIy_z = -(1-n)\frac{z^3}{6} + \frac{(z-nl)^3}{6} + \frac{l^2}{6}(2n - 3n^2 + n^3)z \quad . \quad . \quad (3)$$

Giving n various values between 0 and $\frac{z}{l}$ in equation (3), the value of y_z can be found, and the first portion of the influence line constructed.

(*b*) Let the load be between Z and B, and at a distance nl from A. The deflection at Z for any value of nl greater than z, will be found by adjusting equation (1) by substituting z for x, when

$$EIy_z = -(1-n)\frac{z^3}{6} + \frac{l^2}{6}(2n - 3n^2 + n^3)z \quad . \quad . \quad . \quad . \quad (4)$$

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Giving n various values between $\frac{z}{l}$ and unity, the corresponding values of y_z can be found and the second portion of the influence line drawn.

To find the position of the load to give a maximum value of the deflection, differentiate equations 3 or 4 with respect to n, equate to zero and solve for n.

For a series of concentrated loads crossing the span, the deflection at Z will be $y_z = \frac{1}{EI}\Sigma Wy$

where $$\Sigma Wy = W_1y_1 + W_2y_2 + \ . \ . \ . \ W_ny_n$$

and where $y_1y_2 \ . \ . \ . \ y_n$ are the ordinates of the influence line (unit) at the loads $W_1W_2 \ . \ . \ . \ W_n$.

Therefore the area of the unit deflection diagram above a length of uniformly distributed loading times the rate of loading will give the deflection at the section Z as a uniformly distributed load moves over the span. Depending upon the position and length of the uniform load, the deflection at Z can be obtained by integrating either or both of equations (3) and (4) with respect to nl within the proper limits.

GENERAL CASE.

Let a uniform load of w tons unit length and of a length greater than l cross the span. The maximum deflection will occur when the whole span is covered.

Then

$$EIy_{zmax} = \int y_z w \, . \, d(nl) = wl \int_{n=0}^{n=1} y_z \, . \, dn \text{ (because } l = \text{constant)}$$

$$= -\frac{z^3}{6} \, . \, wl \int_{n=0}^{n=\frac{z}{l}} (1 - n) \, . \, dn + \frac{wl^3z}{6} \int_0^{n=\frac{z}{l}} (2n - 3n^2 + n^3) \, . \, dn$$

$$+ wl \int_0^{n=\frac{z}{l}} \left[\frac{(z - nl)^3}{6} \, . \, dn. \right]$$

$$-\frac{z^3}{6} wl \int_{n=\frac{z}{l}}^{n=1} (1 - n) \, . \, dn + \frac{wl^3z}{6} \int_{n=\frac{z}{l}}^{n=1} (2n - 3n^2 + n^3) \, . \, dn \quad . \quad . \quad (5)$$

The first three terms apply when A to Z covered and the last two when Z to B covered.

Let $z = \frac{l}{2}$

Integrating the first three terms of equation (5) between the limits $n = \frac{1}{2}$ and o, and doubling for maximum deflection when the whole beam is covered by the load

$$EIyz_{max} = 2\left\{ \frac{wl^4}{48}\left[\frac{(1 - n)^2}{2}\right]_{n=0}^{n=\frac{1}{2}} + \frac{wl^4}{12}\left[n^2 - n^3 + \frac{n^4}{4}\right]_{n=0}^{n=\frac{1}{2}} - \frac{wl^4}{384}\left[(1 - 2n)^4\right]_{n=0}^{n=\frac{1}{2}} \right\}$$

$$= 2\left(+ \frac{wl^4}{384} + \frac{9wl^4}{768} - \frac{3wl^4}{384} \right) = \frac{5wl^4}{384}. \quad . \quad . \quad . \quad . \quad . \quad (6)$$

(The general formula from ordinary dead load considerations.)

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The Deflection of a Beam rigidly fixed at both ends and carrying a unit dead load at a distance nl from the left-hand support (the origin) (Fig. 4).

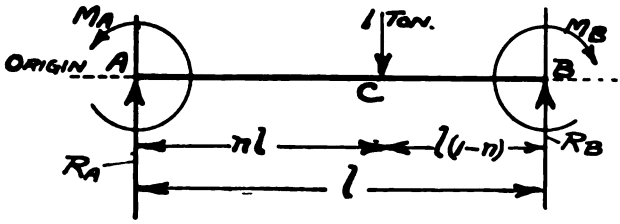

Fig. 4.

Let the beam AB be of length l. Let M_A and M_B be the end fixing couples. From the usual formulæ and theory it can be found that

$$M_A = nl(1 - n)^2 \text{ and } M_B = n^2l(1 - n).$$

For the general case take $M_B > M_A$.

Then R_A = Reaction as for a Simple Beam minus $\dfrac{M_B - M_A}{l}$

$$\therefore R_A = (1 - n) - \left(\frac{M_B - M_A}{l}\right)$$

Substituting for M_B and M_A in terms of n

$$R_A = (1 - 3n^2 + 2n^3).$$

Values of n	R_A	R_B
0	1	0
$\frac{1}{4}$	$\frac{27}{32}$	$\frac{5}{32}$
$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$
$\frac{3}{4}$	$\frac{5}{32}$	$\frac{27}{32}$
1	0	1

Take any section X between the load and B distant x from A.
Then

$$M_x = EI\frac{d^2y}{dx^2} = -(1 - 3n^2 + 2n^3)x + (x - nl) + M_A.$$

$$EIy = -(1 - 3n^2 + 2n^3)\frac{x^3}{6} + \frac{(x - nl)^3}{6} + \frac{nl}{2}(1 - n)^2x^2 + Px + Q.$$

As the beam is rigidly fixed at both ends then both P and Q equal to zero.
For sections between A and C, i.e. for values of x between 0 and nl

$$EIy = -(1 - 3n^2 + 2n^3)\frac{x^3}{6} + \frac{nl}{2}(1 - n)^2x^2 \quad . \quad . \quad . \quad . \quad (7)$$

For sections between C and B, i.e. for values of x between nl and l,

$$EIy = -(1 - 3n^2 + 2n^3)\frac{x^3}{6} + \frac{nl}{2}(1 - n)^2x^2 + \frac{(x - nl)^3}{6} \quad . \quad . \quad . \quad (8)$$

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If $n = \frac{1}{2}$ and $x = \frac{l}{2}$, from equation (7)

$$EIy_{centre} = -\frac{l^3}{96} + \frac{l^3}{64} = \frac{l^3}{192}.$$

If $nl > \frac{1}{2}$, the maximum deflection will occur in the length $AC = nl$.

If $nl < \frac{1}{2}$, the maximum deflection will occur in the length $CB = l(1 - n)$.

The section at which maximum deflection occurs can then be found by differentiating either equations (7) and (8) with respect to x, equating to zero and solving.

(*To be concluded*).

Bond Between Concrete and Steel.

In describing the results of his investigations into the relation between the bond between concrete and steel and the factors which govern the compressive strength of concrete, Professor Duff A. Abrams gave the following as his conclusions to a recent meeting of the American Society for Testing Materials. The tests were made by applying a pull on one end of 1-in. plain round steel bars embedded axially in 8-in. by 8-in. cylinders. The concrete specimens were made from a wide range in size and grading of aggregates, quantity of water, and cement, and were tested at ages from seven days to one year.

(1) A steel bar embedded in concrete offers considerable resistance to a pull-out load. Slipping of the bar begins at a bond stress of about 10 to 15 per cent. of the compressive strength of the concrete, but considerable additional load is taken before the ultimate bond resistance is reached. For pull-out tests of the type used, 0·0005 in. end slip of bar occurred at 55 to 60 per cent. of the maximum bond ; for mixtures lower than 1 : 1, the maximum bond was about 24 per cent. of the compressive strength of the concrete and came at an end slip of about 0·01 in.

(2) Bond and compressive strength increased with age of the concrete from 7 days to 1 year. For 1 : 5 concrete of water-ratio 0·88, the bond at 1 year was 134 per cent. of the 28-day value, and compressive strength 148 per cent.

(3) Bond responded to changes in water-ratio of the concrete in much the same way as compressive strength ; increase in water-ratio due to use of wetter concrete, less cement, or an excess of fine aggregate, resulted in material reductions in both bond and compressive strength. Other tests have shown that the same statement applies to the impermeability, resistance to wear, and resistance to destructive agencies such as weather, sea and sulphate waters, etc.

(4) For mixtures richer than 1 : 1 the bond fell off, probably due to the greater volume changes during hardening which are characteristic of such mixtures.

(5) The use of 4 per cent. of the 28-day compressive strength of concrete as the working stress in bond for plain bars, as specified by the Joint Committee, is justified ; this gives a factor of safety of about 2½ to 3 against first slip.

(6) The use of crude oil to replace mixing water, in general, caused a reduction in both bond and compressive strength of concrete. Five per cent. of oil reduced the bond at 28 days about 20 per cent. ; at 1 year, about 6 per cent. ; compressive strength at 28 days and 1 year was reduced about 3 per cent.

(7) Replacing cement with hydrated lime decreased the compressive strength and bond about 1·2 per cent. for each 1 per cent. of hydrated lime in terms of volume of cement, or about 2·0 per cent. for each 1 per cent. by weight.

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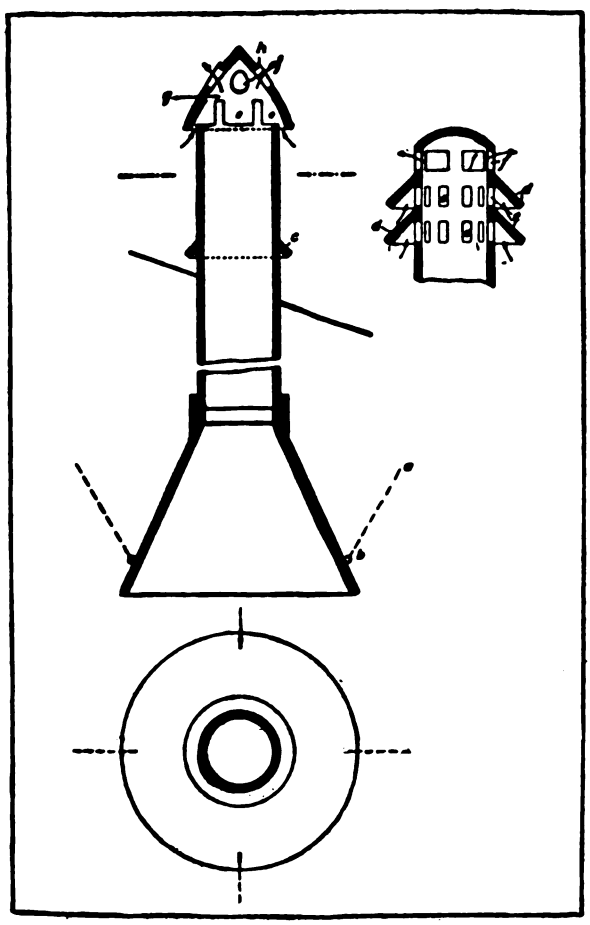
Concrete Smoke-Stacks for Railway Running Sheds

THE short life of sheet-iron smoke-stacks exposed to the sulphurous gases of running sheds is a matter of concern to engineers dealing with railway maintenance. An illustrated article on a new reinforced concrete stack, manufactured by Messrs. Wayss and Freytag, appeared recently in *Tonindustrie-Zeitung*, from which we gather the following particulars.

The stack is composed of three pieces. The base is a truncated cone with a bottom diameter of 4 ft. 7 ins. and a top diameter of 1 ft. into which fits the cylindrical portion of which the walls are 1 in. thick. The conical top-piece has four openings for the escape of the gases. To suit different roofs the makers manufacture the stacks in lengths varying from 10 to 20 ft. A very light concrete is used, and the interior surface is rendered with an acid-resisting composition.

W.S.G.



Concrete Smoke-Stack.

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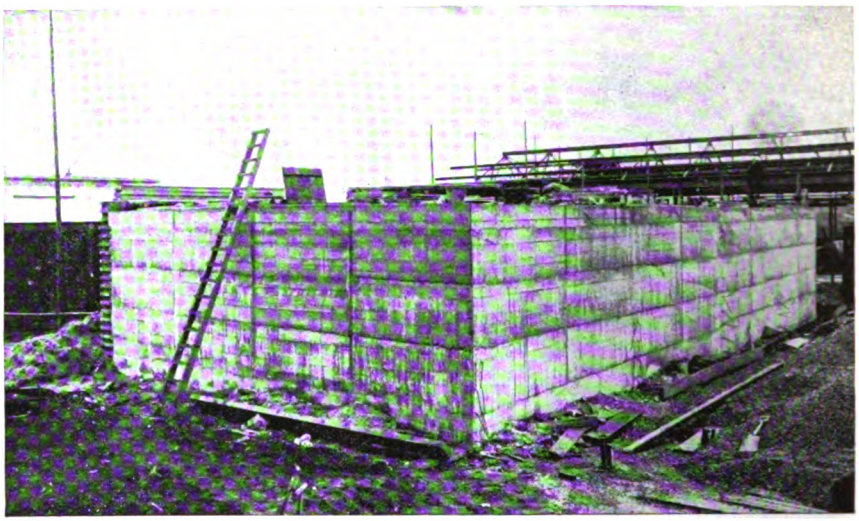
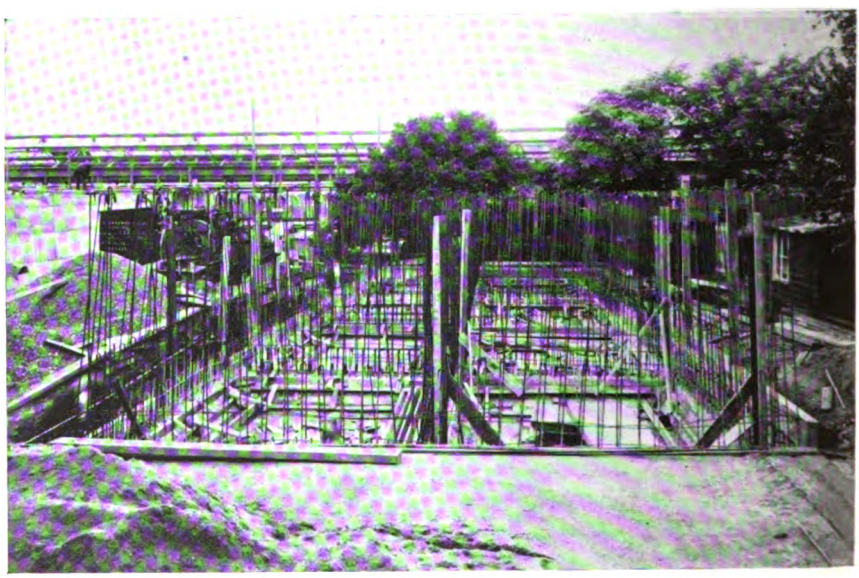


Completed Tanks "A."



Tanks "B": Underground portion during Construction.

Reinforced Concrete Tanks for Storage of Wine. (*See p.* 499.)

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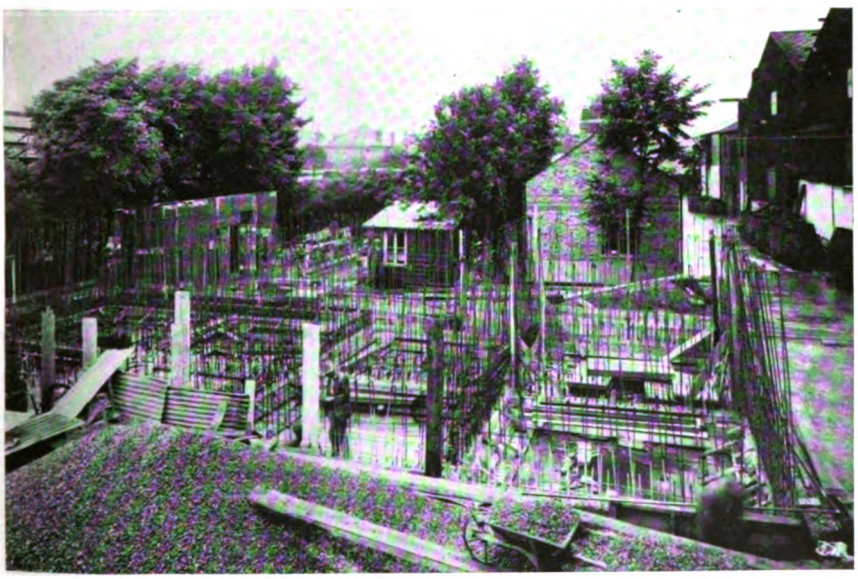
Reinforced Concrete Tanks for the Storage of Wine.

THE tanks illustrated on this and the following pages are being constructed at Kingston-on-Thames for storage of wine. There are altogether sixteen tanks, in groups of eight. They are partly sunk underground, and each tank has a clear capacity of 14,000 gallons, the internal dimensions of the tanks being 16 ft. by 13 ft. by 12 ft. deep. The space between the two groups of tanks is for future tanks, and therefore one external longitudinal wall of each group of tanks has been carried down below the level of the floor to form the wall of future tanks. This is shown on the transverse sections through the tanks (*Fig.* 4).

The walls of the tanks are taken as fixed at the bottom and supported at the top, and are 9 in. thick at bottom and 7 in. at top. The roofs are designed to carry a uniformly distributed load of 6 cwts. per foot super exclusive of the weight of the slab, and have been designed as fixed over the centre division walls and supported on the external walls, so as to avoid bending stresses in the external walls. The reinforcing bars to walls, etc., are shown on *Fig.* 3.

The concrete is composed of 1 part rapid-hardening "Ferrocrete" cement, 2 parts clean sharp sand, and 4 parts clean crushed ballast, the sand and ballast being obtained from the Ham River Company. The use of rapid hardening cement and the promptness with which the aggregates were delivered have enabled the contractors to make very good progress with the concreting. The wall forms were struck in less than twenty-four hours after concreting, and the shuttering to roof slabs in forty-eight hours; as these slabs are of an average thickness of 10 in. the rapid hardening qualities of this cement is fully proved, and had ordinary Portland cement been used such rapid progress would have been impossible.



View of Tanks "B" during Construction.

The tanks have been designed by Mr. T. A. Harvey, consulting engineer, and Messrs. A. Jackaman & Sons, Ltd., of Slough, are the contractors for the works.

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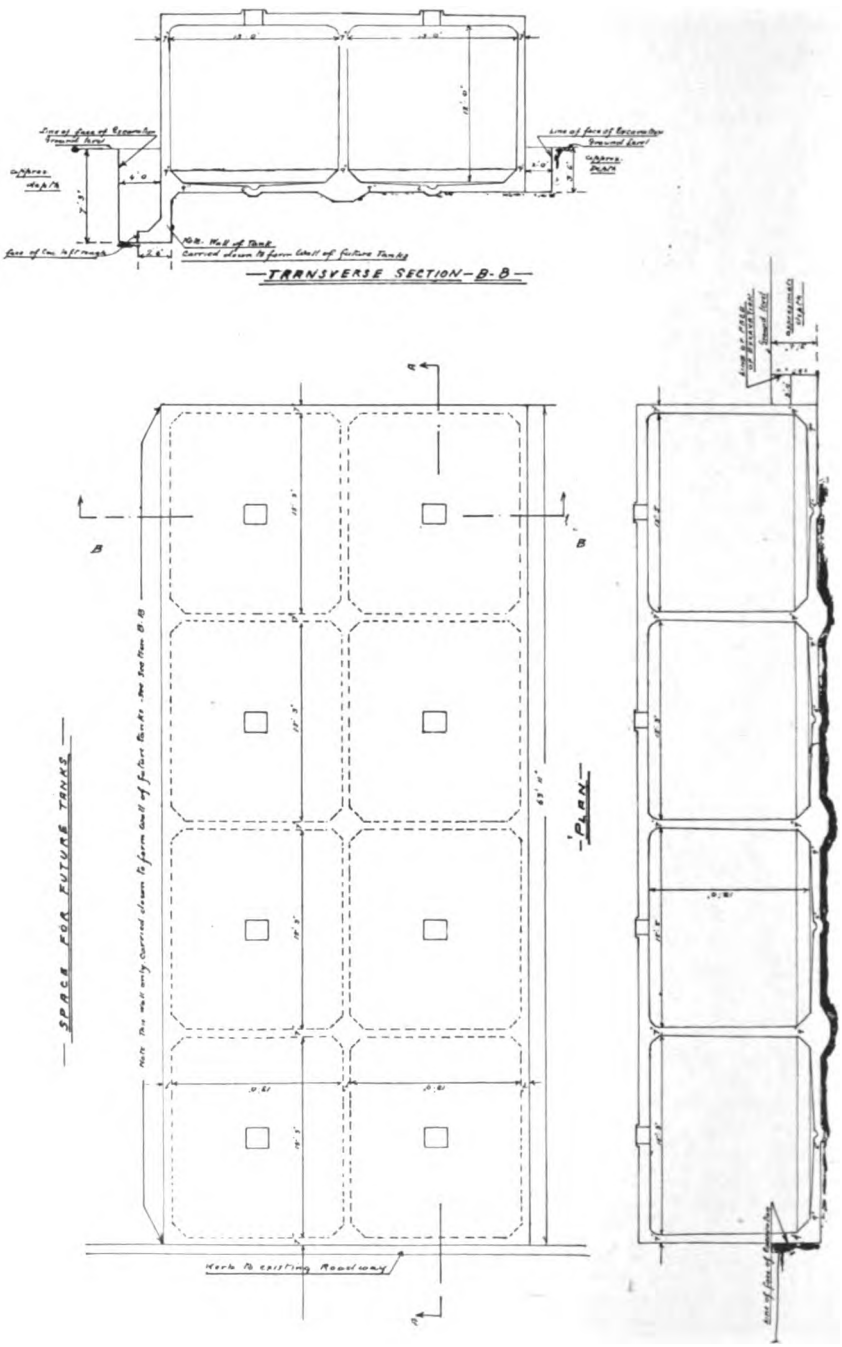

Fig. 2.—Block Plan and Sections of Tanks "B." (See p. 499.)

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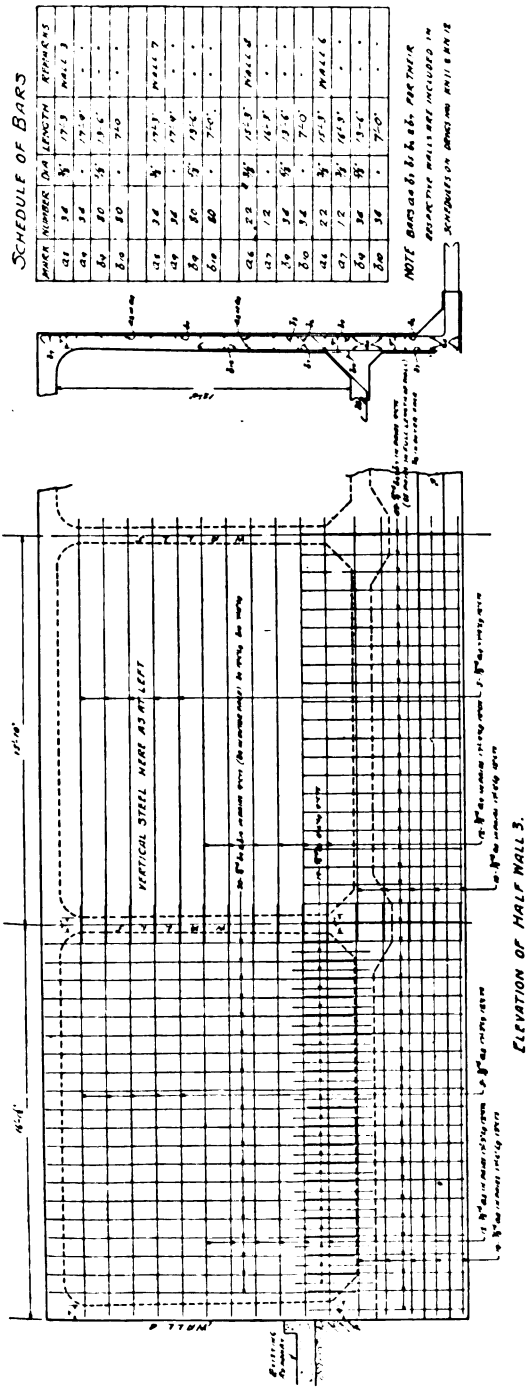

Fig. 3.—Elevation of External Wall of Tanks showing Reinforcing Bars. (*See p.* 499.)

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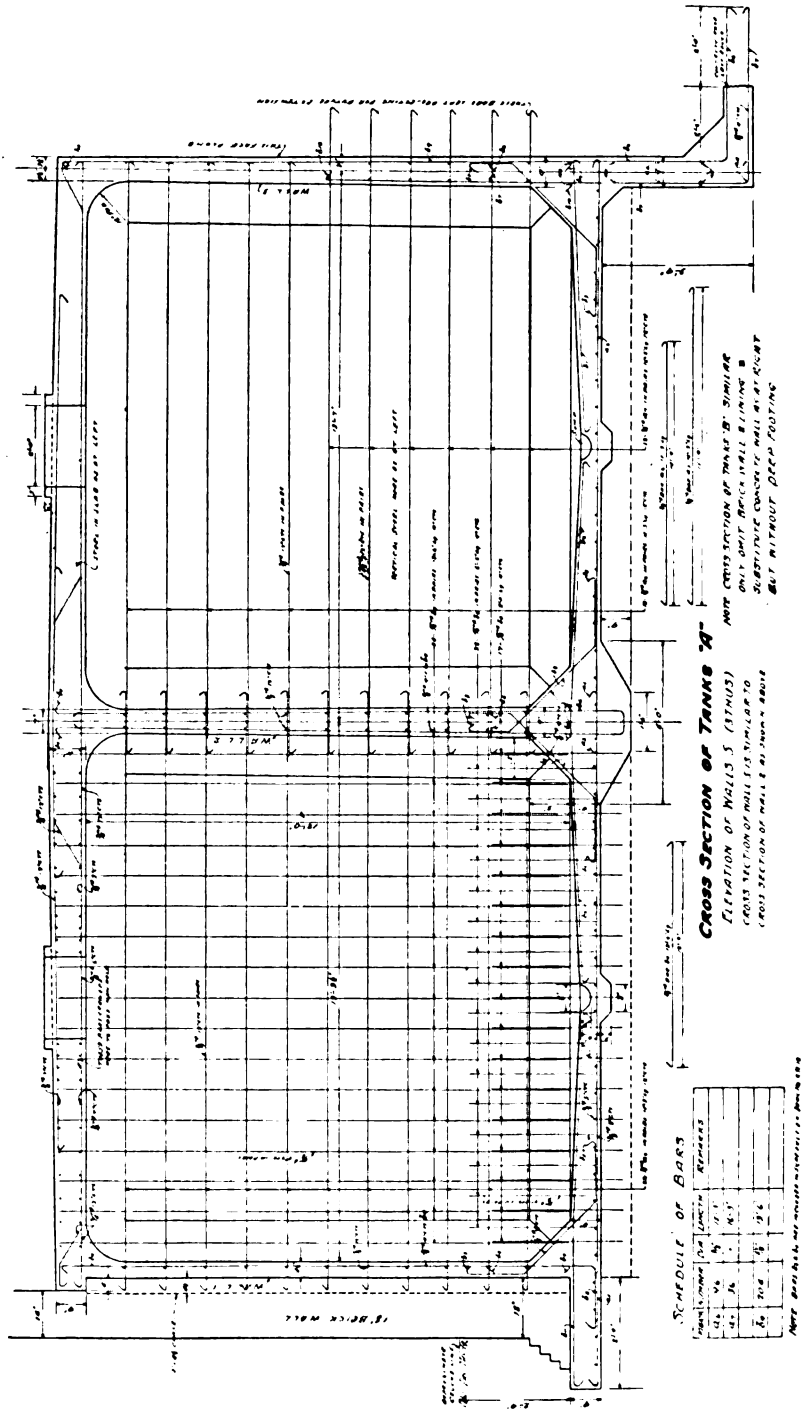

Fig. 4.—Cross Section of Tanks "A." (*See p.* 499.)

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Reinforced Concrete in Perth, Western Australia.

For the illustrations given of the two buildings in Perth, Western Australia, we are indebted to Mr. E. P. Henshaw, of A.M.P. Chambers, Perth, who designed and supervised the reinforced concrete work in both cases. In some notes accompanying the illustrations Mr. Henshaw says:

The new building for the Winterbottom Motor Company, illustrated herewith, is being erected in St. George's Terrace, Perth's finest thoroughfare, and is considered to be the finest of its kind in Australia.

The main features of the building comprise a large garage, with a four-way mushroom floor, with columns at 22 ft. 6 in. and 18 ft. 1 in. centres. The depressed panels are made circular, and the slab itself has an overall thickness of 9 in. The conditions favour such a floor, which is approximately 165 ft. square, as full advantage can be obtained from the continuity of the system. Below this floor are the assembling, painting and trimming shops. There is also a hollow-tile floor over the showrooms, over which are professional and residential chambers. The area covered by this floor is 212 ft. by 65 ft. The secondary beams or joists run lengthwise of the building. The tiles are of coke-breeze concrete and spaced at 18-in. centres, providing filling of the beams to a width of 6 in., in which the reinforcement is placed. The main cross-beams are tee shape in section, with added reinforcement for the internal columns to counteract the reduced stem section and heavy negative moment.

The main floor beams at two points have a bearing over the large show window, to which heavy loads are transmitted. Ample provision has been made by a lintel with a section of 32 in. by 16 in., reinforced to take these concentrated loads. All columns were constructed with 18 in. Hume pipes as a casing, the reinforced concrete filling being capable of taking the entire load and bending stresses irrespective of the value of the casing. While the walls and piers are of brick, at each floor and ceiling level a reinforced concrete lintel is carried right around and through the building, giving considerably added stability to the structure.

The work is being regarded with much interest locally, as both systems of construction are new to the city.

The architects for the work are Messrs. Oldham, Boas & Ednie-Brown; the contractor for the work was Mr. C. W. Arnott.

The building for the Young Australian League contains a large gymnasium and concert hall, over both of which it was necessary to span floors and ceilings without the obstruction of columns. The

New Premises for the Winterbottom Motor Co., Perth.

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"Mushroom" Floor under Construction.

"Mushroom" Floor under Construction.

Showing Marginal Steel: 4-Way "Mushroom" Floor.

New Premises for the Winterbottom Motor Co., Perth.

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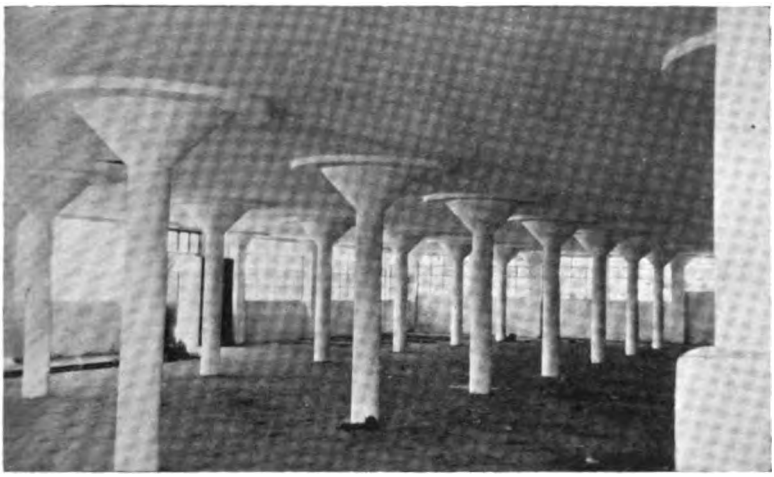

New Premises for the Winterbottom Motor Co.: Part of "Mushroom" Floor.

spans approximate to 60 ft. The original design embodied steel beams, but during construction difficulty arose in connection with the delivery of the steel girders, and recourse was had to reinforced concrete beams. A double reinforced type was adopted, and the necessary bar steel being available the beams were placed without delay. The section of the beams, four of which were erected, was 48 in. by 27 in., and the length 60 ft. The soffits and sides were recessed slightly to form panelling. The total load provided for was 248 lbs. per sq. ft. of floor. The cost of the work was approximately the same as for the steel originally provided. The architects for the building were Messrs. Oldham, Boas & Ednie-Brown.

Young Australian League Club Room, Perth.

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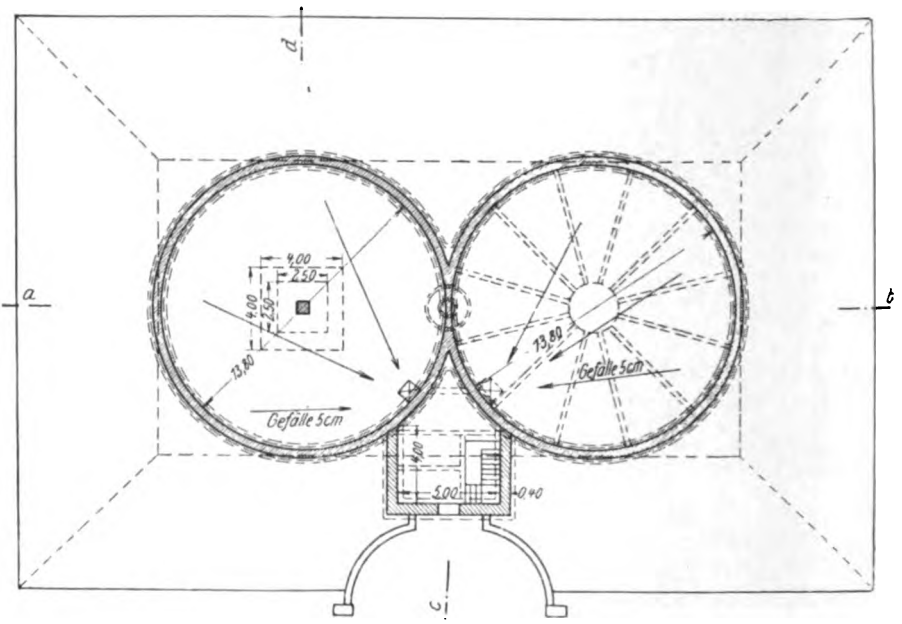
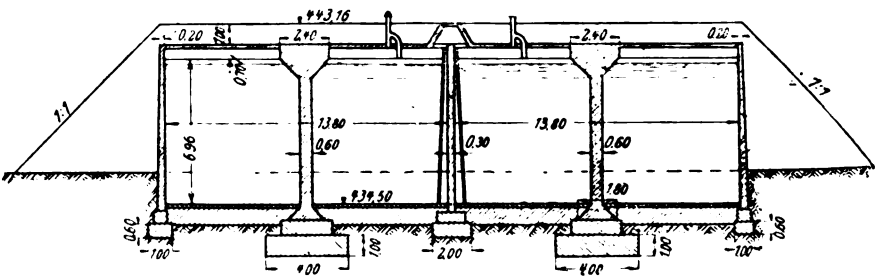
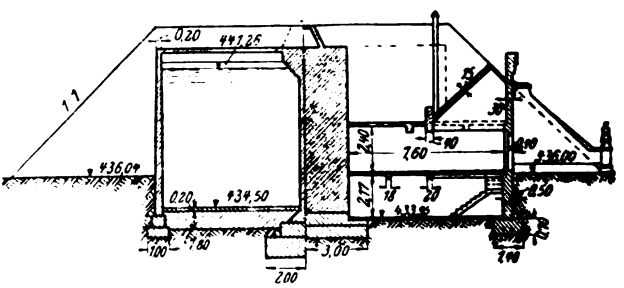


Plan.



Section A—B.



Section C—D.

A 550,000-gallon Water Reservoir. (*See p.* 507.)

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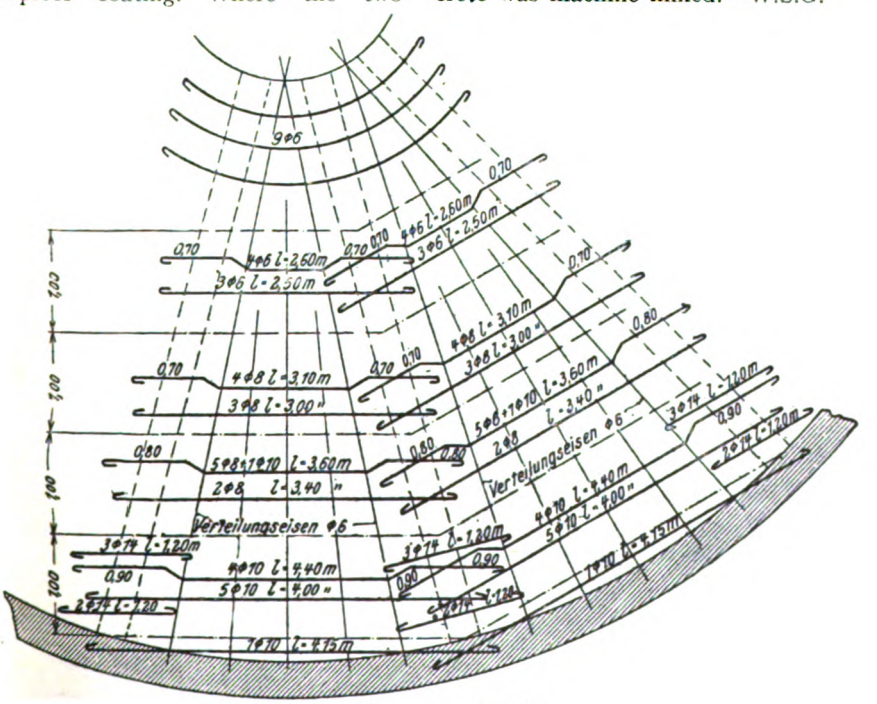
A 550,000-gallon Water Reservoir.

THERE has recently been constructed at Rothenburg a tank in reinforced concrete capable of storing 550,000 gallons. It was decided to hold a competition for a design and to give the competing firms a free hand so far as the form and construction of the tank were concerned. The only restrictions imposed were, (*a*) a capacity of 550,000 gallons; (*b*) the tank to be in two compartments; (*c*) the tank to be covered with 3 ft. 3 in. of soil; and (*d*) the design to conform to the local regulations for reinforced concrete. The design of Messrs. Dyckerhoff and Widmann was accepted, and the work put in hand.

The design consists of two equal cylindrical tanks, each of 45 ft. diameter and 28 ft. high, arranged so that they have a common wall for a length of 13 ft. The soil taken out for the foundations was used to cover the tank to a depth of 1 ft. all over and with one-to-one side slopes. The floor of the tank is composed of a layer of lean concrete 2 ft. 7 in. thick, spread over the sub-soil. Above this is an 8-in. layer of richer concrete with a waterproof coating. Where the two tanks meet there is a common manhole with openings into both tanks, and an air-inlet pipe is concreted into the roof of each compartment. Our illustrations and description of this work are taken from *Beton u. Eisen* of a recent date.

In the design the allowable ground pressure had to be limited to 1·65 tons per sq. ft. The walls are reinforced against hoop stresses and have vertical rods in addition. In determining the area of tensile steel required the tension taken by the concrete was neglected and an allowable steel stress of 14,200 lbs. per sq. in. used. The thickness of the wall was so arranged that the concrete stress should not exceed 142 lbs. per sq. in. No allowance was made for partial fixation between the wall and the floor or roof. A number of radial beams running from the walls to the central support and continuous over the latter divided the roof slab into sections, which were reinforced as shown in the illustrations.

The inner surface of the walls and the upper side of the roof were treated with a waterproof plastering coat. All the concrete was machine mixed.—W.S.G.



Reinforcement of Slab.

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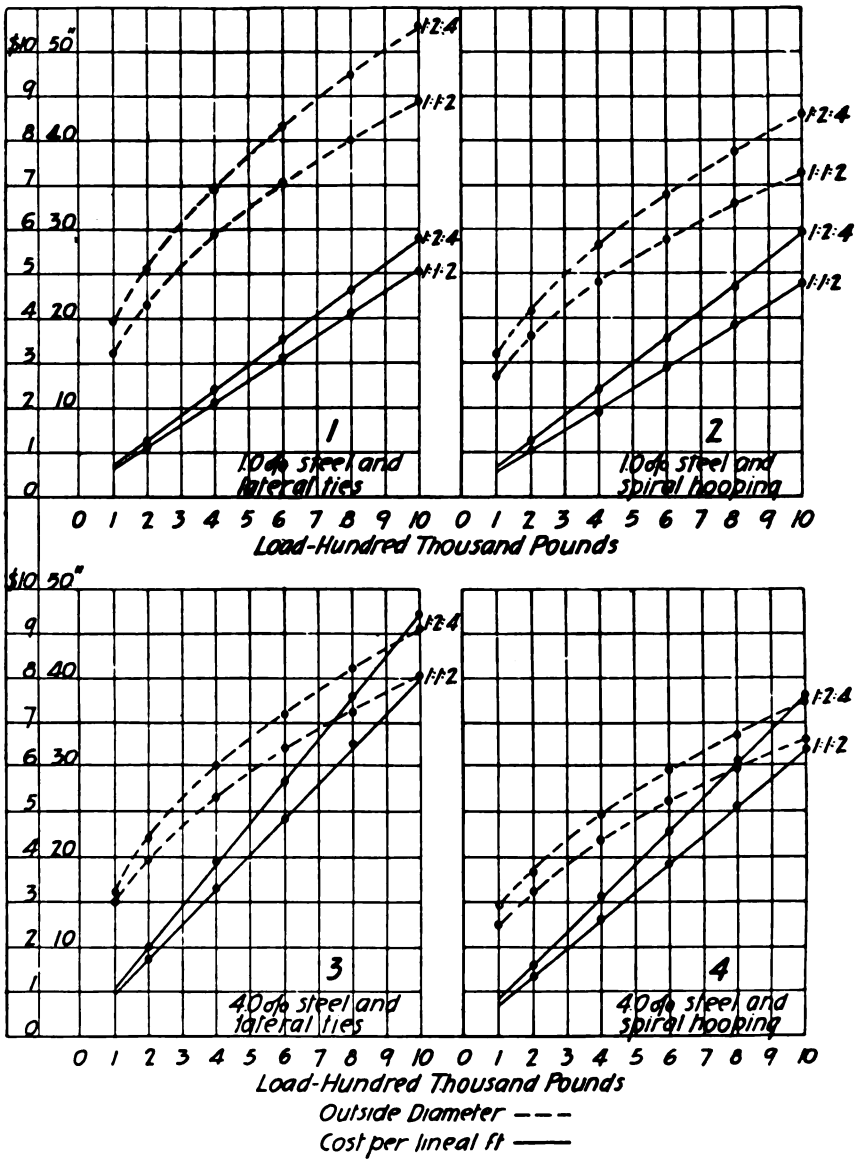

Curves showing Effect of Richer Concrete on Cost and Size of Round Reinforced Concrete Columns. (*See p.* 509.)

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Cost of Rich and Lean Concrete Columns.

In a recent issue of our American contemporary, *Concrete*, Professor J. H. Thompson gives some curves (reproduced on p. 508), showing the relative cost of reinforced concrete columns made with rich and lean mixtures. The proportions used for the purpose of the investigation were 1 : 2 : 4 and 1 : 1 : 2, and the figures are based on present day prices for cement, aggregate and reinforcement. The curves are based on the total cost of materials per lineal foot of round columns with each mixture, and for high and low percentages of steel and for both vertical steel combined with ¼-in. lateral ties spaced 1 ft. apart and for vertical steel with 1 per cent. of spiral hooping by volume. The concrete strength is reckoned apart from 1½-in. of fireproofing allowed in each case.

From these curves it will be seen:

(1) The cost of column per lineal foot is greater for the leaner concrete, everything else, including the workability of the concrete, remaining the same. This increase is relatively greater for the higher column loads, and, taking 500,000 lb. as the average of the load range studied, amounts to from 15 per cent. for the most favourable condition (Curve 1), to 24·5 per cent. for the most unfavourable condition (Curve 2), an average for the four conditions of 19 per cent. on material cost.

(2) The diameter is greater for the leaner concrete, everything else, including the workability of the concrete, remaining the same. The increase in column area at the average load of 500,000 lbs. ranges from 26·5 per cent. for the most favourable condition (Curve 4), to 41 per cent. for the most unfavourable condition (Curve 1), an average for the four conditions of 34 per cent.

Attention is also directed to the following interesting facts which have, however, no direct bearing upon the question under discussion: (*a*) Cost is directly proportional to load; (*b*) for the same percentage of vertical steel, columns employing spiral hooping are cheaper than those employing lateral ties; (*c*) diameters increase much more slowly as loads increase.

The Santa Barbara Earthquake.

Reinforced Concrete Buildings Undamaged.

In a description of the result of the earthquake, which occurred in Santa Barbara, California, in June, which Mr. Henry D. Dewell, a member of the committee of engineers which surveyed the damage and reported thereon, contributes to "Engineering News-Record," he states that the well-designed reinforced concrete buildings successfully withstood the earthquake, and remained undamaged.

He states that the great extent of the damage to the buildings in the town was due largely to poor design and construction, and this included reinforced concrete frame buildings where only a light frame, poorly designed, was used. Modern reinforced concrete buildings which were well designed, well tied together with cross walls of reinforced concrete, and built of good materials and workmanship, suffered no material damage. In this case there was no opportunity to compare the effect of the earthquake on steel-frame and reinforced concrete buildings, as there were no steel-frame buildings in the town. In the buildings constructed with a light reinforced concrete frame those which apparently had no attention paid to bracing for resistance to lateral forces suffered severely. In all the light reinforced concrete frames which were damaged the effect of the bending of columns was particularly noticeable, and marked by the crushing of the concrete at the top of the first-floor columns. In some cases the steel rods were buckled, and in many cases the reinforcement was exposed. No attention appeared to have been given in the design to resistance against lateral forces which are always exerted by earthquakes, and the columns —being weaker in bending than the beams—suffered greater damage; no fillets or gussets were used.

In his conclusions, the author expresses the opinion that reinforced concrete frames without solid reinforced concrete walls, and without adequate bracing against lateral forces, are not suited to resist earthquakes; and that reinforced concrete of good quality, properly designed, came through the shock without showing more than minor damage.

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Warwick Road Bridge, Stretford.

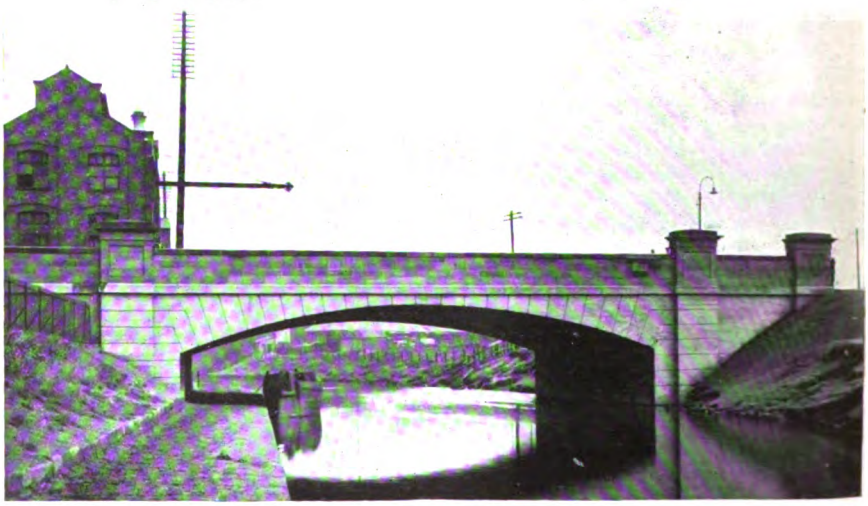
An interesting example of bridge construction has recently been completed at Stretford, near Manchester, in order to provide new means of communication between Trafford Park and Chester Road, one of the chief arteries along which an immense volume of traffic between Manchester and Cheshire now flows. The need for access to Trafford Park at its south-eastern corner has been urgent for several years, and doubtless would have been supplied long ago but for the fact that the Bridgewater Canal runs between the Park and the Chester highway. Warwick Road, which extends from the highway to the canal, has now been adapted to new traffic requirements, and has been carried forward over the canal by a bridge which will henceforth be known as Warwick Road Bridge.

The bridge, which has been built for the Stretford Urban District Council, is not exceptional in span or type, but it is interesting by reason of the manner in which the designers have adapted it to exceptionally heavy traffic—a problem with which bridge builders are more and more called upon to deal. The load for which the bridge has been designed is one vehicle of 70-tons weight on two axles, with a wheel-base of 10 ft. by 6 ft., the load on each wheel being not more than 18 tons.

The bridge is of the arched type, its main dimensions being: clear span, 60 ft.; rise from the springings to the soffit of the arch at the crown, 5 ft. 6 in.; width between the outside faces of the arch, 53 ft. 8 in. The height from the level of the canal water to the arch soffit at the crown is 13 ft. 6 in.; and the head room at the springings down to the level of the 10-ft. tow-path is 7 ft. The small rise of the curve gives the arch a rather flat appearance, but this was deliberately contrived in order to avoid encroaching on the head room to the canal below and making an excessive gradient on the road above.

There are no ribs in the arch. It is a pure segmental slab construction, 16 in. thick at the crown and 24 in. thick at the springings. Owing to the heavy loads it must carry the main arch reinforcement is rather large in section, and consists of $1\frac{1}{8}$-in. diameter bars at 10-in. centres in both the top and bottom of the slab. Each bottom bar is tied to the corresponding top bar by B.R.C. stirrups.

Since it was essential to spread the load over as great a width as possible, the method of calculation permitted under



[*Mr. E. Worrall, Engineer to the Urban District Council.*

New Reinforced Concrete Bridge at Stretford.

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the French Government Regulations was adopted, and this accounts for the comparatively large section of transverse reinforcements, namely, ¾-in. diameter bars at 9-in. centres in the bottom of the slab, and ⅝-in. diameter bars at 18-in. centres in the top. As the bridge passes over the Bridgewater Canal—which is controlled by the Manchester Ship Canal Co., Ltd.—the design had to receive this Company's sanction and their standards of working stresses had not to be exceeded. These stresses, which were accordingly adopted in the design, are:

	lbs. per sq. in.
Compression in concrete (maximum allowable)	500
Tension in bars	15,000
Tension in B.R.C. fabric	20,000

The abutments, which are similar on each bank, consist of seven deep buttresses joined together by a vertical front wall 18 ft. high. This wall, therefore, has six bays of approximately 8 ft. 9 in. each. The top of the wall at the arch springings is surmounted by a horizontal beam which spans between the buttresses and distributes the arch thrust through the buttresses to the base slab. Each bay of the wall is pierced by one weep hole.

The base, or foundation slab, which at the bottom of the front wall is 10 ft. below the canal water-level, runs back horizontally from this point for a distance of 8 ft. and then inclines upwards at an angle of 29 deg. for a length of 22 ft., and terminates in a heel beam. The inclined portion was so arranged as to spread the resultant pressure evenly, the centre of pressure occurring at the middle of the 22-ft. section.

The top surface of each buttress forms a straight line from the heel beam up to the thrust beam, at which point it runs tangentially into the top surface of the arch slab. On each side of the bridge short wing walls are constructed. Those on the north side are continuous with and parallel to the parapets. On the south side the walls are slightly curved so as to be in line with the roadway, which approaches at an angle. Each wing wall, therefore, rests upon an end abutment buttress, and is prevented from overturning by counterforts that run back to the next abutment buttress.

The parapets are of masonry resting on a reinforced concrete stringcourse, which is carried on reinforced concrete spandril walls cantilevered from the arch slab. The height of the parapets is 4 ft. from the footway level to the coping. At the ends of the bridge and over the wing walls stone piers are inserted, the height from the footway to the top of the caps being 7 ft. 6 in. The outside faces of the spandril walls have been effectively treated by grooving them to give the effect of voissours, and this, in conjunction with the substantial piers, gives an appearance of solidity and strength.

At the crown of the bridge the distance from the concrete to the road surface is only 8 in. This, of course, increases towards the springings, at which point the filling is 5 ft. 6 in. deep. The width of the carriageway is 34 ft., and this allows for an 8 ft. 6 in. footway on each side.

Work was begun on the site in early spring, the first excavation being for the south abutment. Owing to the proximity of the canal, and the depth below water-level to which it was necessary to excavate, a cofferdam had to be constructed along the whole length of this abutment. This was built of timber and steel sheet piles and packed with clay. It proved very satisfactory, only a little pumping being required occasionally. On the north side the position of the tow-path made this method unnecessary, and all that was required was a line of steel piling on the land side of the tow-path. The ground at foundation level consisted of good hard clay which was considered strong enough to withstand the calculated pressure of 2·4 tons per sq. ft.

The first concrete was poured early in April, this being the base slab of the south abutment. From this time the work proceeded steadily until the completion of both abutments in mid-summer.

Constructing the arch was difficult owing to the necessity for keeping a generous fair-way in the canal for the heavy barge traffic. Two rows of heavy timber piles were driven into the bed of the canal so that an opening 20 ft. wide was left. The piles were protected by longitudinal fenders, the fender on the tow-path side having a special gangway above to enable the tow-lines to be carried through the opening. The piles were connected at the top by timber beams which ran parallel to the canal.

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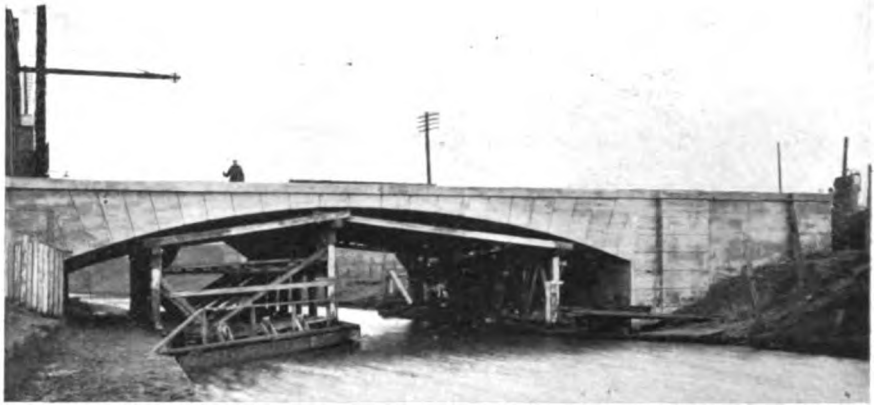

[*Mr. E. Worrall, Engineer to the Urban District Council.*

Reinforced Concrete Bridge at Stretford: Under Construction.

These beams carried 6 in. by 5 in. steel joists at 3 ft. centres, laid with the web segmental. The whole length of each joist was curved to conform with the underside of the arch, four lengths in each joist being used to cover the full 60-ft. span. In the top channel formed by this positioning of the rolled steel joists were placed short lengths of timber, and supported on these in a transverse direction were 4 in. by 2 in. wood joists spaced at 18 in. centres. These wood joists carried the final boarding. The formwork and propping described was only taken over half the width of the bridge, so that one half the width of the full span could be concreted and then, after a suitable time had been allowed for setting, the centering could be struck and re-erected for the remaining half.

The concreting of the arch slab began at the end of September, the concrete being deposited in sections so placed that no undue or eccentric pressures or thrusts would be set up in the formwork. For instance, the two first batches were laid in suitable widths extending from each of the abutments for a length of 5 ft., the next batch then being laid at the crown over a length of 8 ft. The remaining portions were then completed in a number of variously disposed strips. This arrangement proved satisfactory, no appreciable deformation of the formwork being observed at any point. The mixture of concrete was 1 : 2 : 4, this being prepared in a mechanical mixer situated near the bridge and conveyed to the required position in barrows. Ample time was allowed for the setting of the concrete in all cases, and this, together with the fact that a good deal of frosty weather was experienced—during which it was considered safer to suspend all concreting operations—lengthened the period of construction until the early part of the following year.

The upper surface of the arch slab was covered with an asphalte finish, as were the inside faces of the spandril walls up to the roadway level. Shortly afterwards the masonry parapets and piers were erected, and the bridge finally completed.

The contractors were Messrs. W. H. Worthington, Ltd., Manchester. The design was prepared, and all the reinforcing was supplied, by the British Reinforced Concrete Engineering Co., Ltd., Manchester. The design and construction were under the supervision and subject to the approval of Mr. E. Worrall, Engineer to the Stretford Urban District Council.

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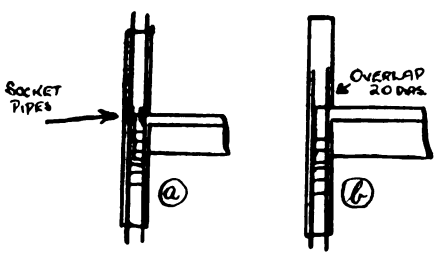
Questions and Answers relating to Reinforced Concrete.

Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.—ED.

Bending in Columns.

QUESTION.—In several publications Dr. Faber has analysed bending in columns. With regard to external columns, I should like to know the method of reinforcement to form the proper degree of fixity which should be present at both ends where the latter terminate at the junctions with floor beams. I am of opinion that sketch *a* would always be ruled out, and that with such connections there is no fixity. I should like your views on this point.—W. G. M.

ANSWER.—We quite agree that the



connection of column rods (*a*) with butt joints connected only by loose socket pipes should always be ruled out. It is, in fact, unsatisfactory from every point of view, even for taking pure compression, as the rods are almost bound to have only point of contact or line concontact, in which case there is sure to be a considerable yield before the rods take their proper share of load. The only proper connection of rods in a column is (*b*) an overlap of at least 20 to 30 diameters. Where there is considerable tension in the rods, 30 or even 40 diameters overlap is desirable.

Waterproofing Concrete.

QUESTION.—Will you kindly advise me of a simple method of stopping a considerable head of water from percolating through the interior surfaces of inferior quality concrete walls lining underground pits? I have had this problem on several occasions, and in each instance the concrete has been of inferior quality, often due to earth falling in while concreting was proceeding.—H.M.

ANSWER.—As a rule the best method of making porous concrete walls in underground pits watertight is to render them with a mixture of two parts of sand to one part of cement. If the water is coming through too rapidly to allow the rendering to be applied, then it is desirable either to relieve the pressure at the back of the pit by drilling occasional holes and letting the water come through in these places and draining it away, blocking the holes up again as soon as the rendering has set hard; or, alternatively, pumping cement grout, or water containing oatmeal, through the holes into the space behind the walls and so stopping the percolation of water. If by this method the percolation can be temporarily stopped and the inside surface kept dry, even temporarily, then the application of hot paraffin wax on the inside surface, spread if necessary with a blow-lamp, is also in many cases a very effective treatment.

Efflorescence on Concrete.

QUESTION.—I am troubled with efflorescence on a large surface area of concrete. Can you suggest a remedy?—J. P. T.

ANSWER.—Efflorescence is, of course, caused by the absorption and evaporation of water, and the problem is therefore to eliminate absorption. This can generally be effected by the application of sodium silicate or magnesium fluosilicate, both of which are colourless. Before applying these washes it is essential to remove all trace of efflorescence, and this is best done by brushing with a stiff brush; if this is not possible with the use of clean water only, use a solution of 1 part of muriatic acid diluted with 8 parts of water, or stronger if necessary. It may be necessary, if the case is an obstinate one, to repeat the application of the hardener two or three times before the efflorescence is finally eliminated.

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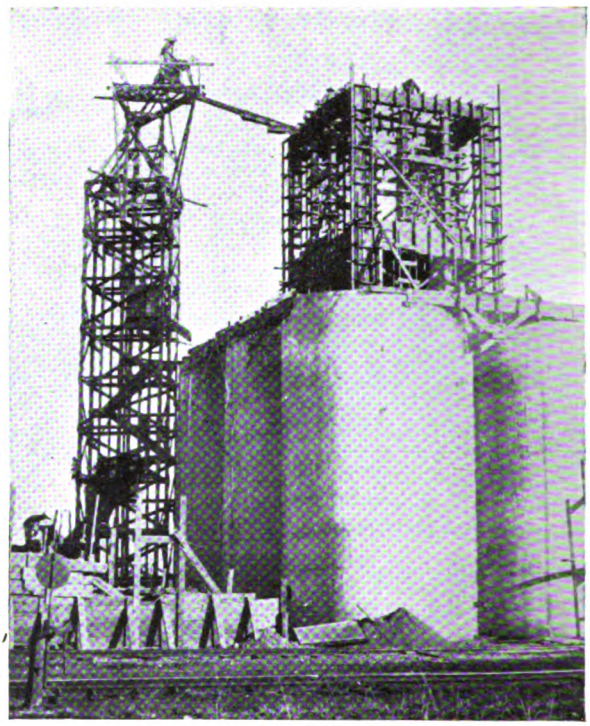

FIG. 2.—DURING CONSTRUCTION.

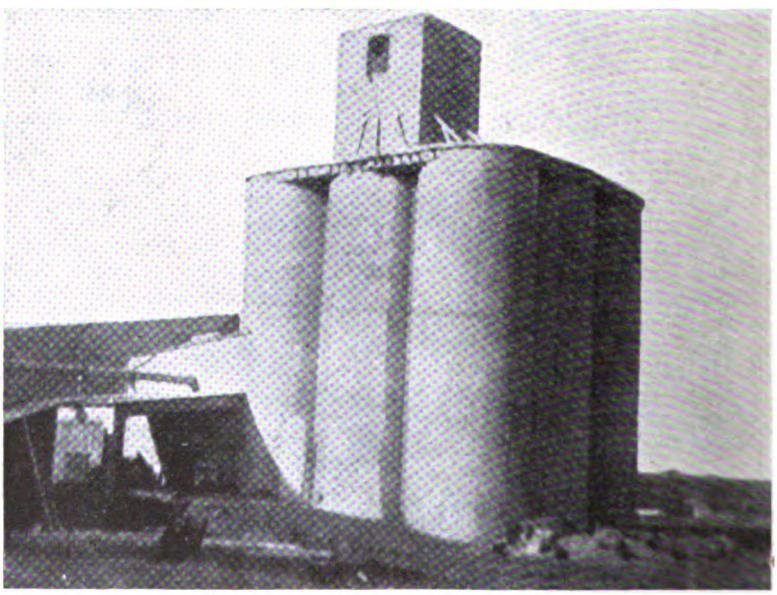

FIG. 1.—COMPLETED.

REINFORCED CONCRETE SILOS IN AMERICA. (*See p.* 515.)

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REINFORCED CONCRETE SILOS IN AMERICA.

A REINFORCED concrete elevator at Wilbur, Washington, is illustrated in *Fig.* 1, while *Fig.* 2 shows the cupola and concrete bins of 81,000 bushels capacity under construction.

The use of special moulds in the construction of reinforced concrete grain bins of 30,000 bushels capacity in eight days at Davenport, Washington, is illustrated in *Fig.* 3, which indicates the method of construction with this system as largely used in the West.

It is maintained that the new moulds used on these structures greatly simplifly concrete wall construction. In the construction of a large concrete elevator for the Great Northern Railway Co. at Superior, Wisconsin, several years ago, in building the concrete bin walls, movable forms were used which were raised with the well-known toggle jack, which is used extensively. It was observed that the raising devices could be moved in a great many ways, and a device was developed which not only gave more satisfaction but, it is claimed, reduced the cost of manipulation by 50 per cent.

This apparatus consists of a wheel, split collar, threaded sleeves, guide plate, two sets of dogs, and an eccentric pawl and U-bolt. One-and-a-quarter inch piping is used to lift the moulds on, and these pipes also act as vertical reinforcing for the concrete walls. These parts are mounted on a yoke, and a graduated rule is attached to one of the vertical yoke timbers with the zero end at the top. Before starting to raise the shuttering the guide plate is brought to zero and as soon as the wheel is turned counter-clockwise the upper dogs automatically grip the pipe, and the yoke and form start travelling upwards, being forced up by the threaded sleeve which is held stationary by the upper dogs.

When the upper end of the sleeve is reached and there are no more threads to climb on, all that is necessary is to reverse the wheel and the lower dogs immediately and automatically grip the pipe and hold the form stationary against downward movement. The upper dogs also release simultaneously and the sleeve and upper dogs are again brought to zero. The device forms a continuous jack which only requires a turn of the wheel in either direction.

When the concrete has set sufficiently hard in the first course the U-bolt is released with the eccentric pawl which releases the outer form, the inner form being built with a $\frac{3}{16}$ in. drift to prevent binding. When raising forms while the concrete is still plastic, releasing of the form is not necessary.

The Wilbur concrete elevator consists of eight circular bins and four interstice

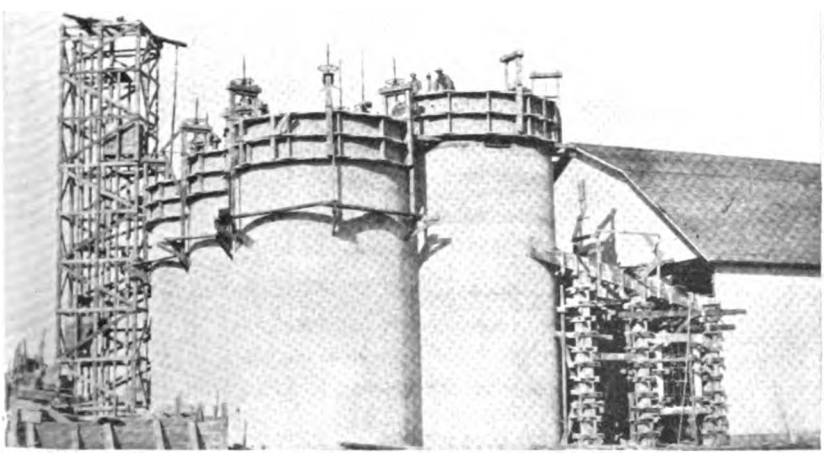

FIG. 3.—SHOWING USE OF CLIMBING MOULDS.

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bins, and required a crew of eight labourers for raising forms, two labourers for wheeling sand, three labourers for wheeling gravel, and a man for hoisting concrete. In addition there were required a labourer at the mixer, another at the chute, three men wheeling concrete, two men spading concrete, and four labourers placing reinforcing steel.

The bins are 16 ft. in diameter with 7-in. reinforced concrete walls. Working ten hours a day this organisation carried the twelve bin walls up 13 ft. in two days and three hours, which was the last 13 ft. of a 55-ft. structure. The forms used were 4½ ft. in depth. The bin walls were carried up 5 ft. a day, working 6 ins. above the concrete which was poured in the morning of the same day. The form came away with such smoothness that the green concrete was not disturbed in the least.

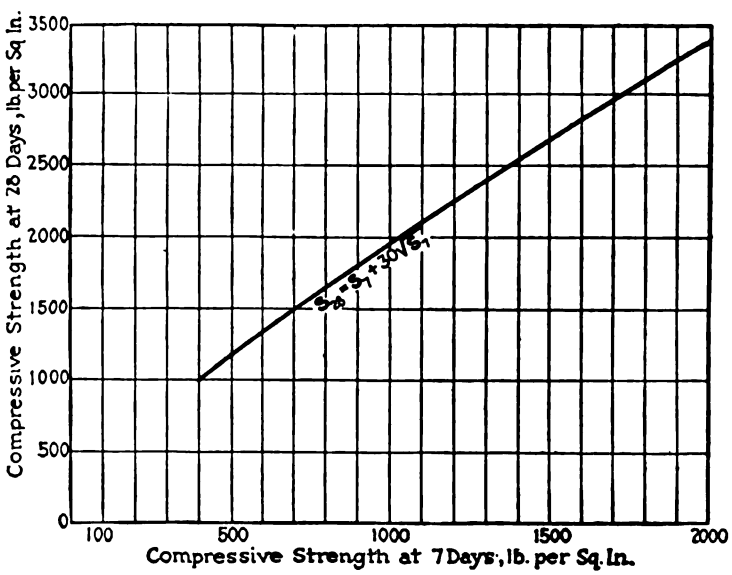
On this concrete elevator the device eliminated eight raising men, two gauge men, an instrument man and rodman, three wrenchmen and three trowel men as compared with the old devices used at Superior.

The Davenport concrete elevator consists of five 16-ft. circular and 2 interstice bins, 30 ft. in height. In spite of the cool nights, these walls were completed within eight days after the forms were placed on the foundation. It is claimed that the same device can be used to great advantage in constructing silos, smoke stacks, stand pipes and water tanks. They can also be used for constructing any kind of concrete walls of varying thickness.

Strength of Concrete at Seven and Twenty-eight Days.

A SERIES of tests undertaken by the United States Bureau of Standards shows that the strength of 28-day concrete is equal to approximately the 7-day strength plus thirty times the square root of the 7-day strength. The samples used in the tests were made of a large variety of different mixtures and with different Portland cements, and the relation of strengths was found to hold good in every case. The equation suggested above is represented in the curve below.

In a bulletin describing the tests, the Bureau points out that this application to the testing of concrete on the job should not be confused with the problem of determining in advance, from suitable tests, the adaptability for the work of the materials proposed to be used, and does not remove the necessity for such tests.



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Design of Formwork for Reinforced Concrete Construction.

By A. E. Wynn, B.Sc., A.M.Am.Soc.C.E.

IX.—DETAIL CONSTRUCTION OF BEAM AND GIRDER FLOOR FORMS—(*Continued*).

Stripping.—The length of time the forms must be left in place before stripping depends on so many conditions, often peculiar to a particular job, that no definite rules can be given, and it is a question that should be decided by the engineer or architect. For an ordinary job, with no special features, the temperature of the air is the most important point. The setting qualities of the cement, the span of the member, and whether the member is under compression only or is subject to bending, are also important points. Long spans should be left longer than short spans, and beams and slabs longer than columns and walls. As a guide the following is given, but it must be combined with judgment. Summer temperature will be taken as 60 deg. Fahr., and winter temperature as 40 deg. Fahr.; for intermediate temperatures the time can be taken accordingly.

Girder and beam sides can be stripped in			3–4	days	with	temp. at	60	deg. F.
			,, 9–10	,,	,,	,,	40	,,
Slabs up to 6 ft. span	,,	,,	,, 5–6	,,	,,	,,	60	,,
			,, 12	,,	,,	,,	40	,,
Beam and girder bottoms	,,	,,	8–10	,,	,,	,,	60	,,
and long-span slabs	,,	,,	21	,,	,,	,,	40	,,

These times are conservative. Below 40 deg. it is mainly a question of the heat and protection given during and after pouring.

All beams must be reshored immediately until the concrete has been poured for at least 28 days, as they must support the construction on the upper floors as well as their own weight.

The order of stripping after the column forms are removed depends somewhat on the method of framing and whether the bottoms are to remain after stripping the slab. If the bottoms and shores are to remain in place the order of stripping is as follows: ribbons holding beam sides; bevelled keys or cleats at junction of beam and girder; ledgers; joists; girder sides; beam sides; floor panels; beam bottoms and shores; girder bottoms and shores. This means that all the forms except the bottoms are stripped together. If the sides are required first and the slab panels left in place a little longer, each shore in turn is loosened, turned around with the cap parallel to the beam, and wedged up again, then the girder sides first and the beam sides next can be taken down, leaving the panels in place and putting temporary shores under them.

If the slab panels have been built with loose or lightly-nailed boards all around the outside they can be removed with the joists before stripping the sides.

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If loose joists with a continuous ledger are used the ledger must be taken off first, but if the joists are loose and carried on blocks they can be knocked off without removing the blocks.

If the panels are nailed to the joists the ledgers or blocks must be removed first unless the shores are turned around and the beam sides removed.

If the bottom and sides are built as a unit the order of stripping will then be : ribbons ; cleats and keys ; shores under girders ; girder sides and bottom together ; shores under beams ; beam bottom and sides together ; and lastly the slab panels.

All beams and girders must be immediately reshored and temporary shores should be placed under the slab panels to prevent them falling while the beam boxes are being removed.

When girder sides are built as in *Fig.* 44, usually the lower half of the sides and the bottom are stripped first, then the girder sides, beam sides, and slab. Small pieces, like battens and cleats, which have to be removed should be tacked on the form to prevent loss.

When the bottoms are left in, the shores and bottom are stripped together and not the shores separately. Shore braces are removed before beginning to strip but are replaced on the permanent shores.

A light scaffold must be built for removing the cleats, ledgers, ribbons, etc. The beam sides and slab panels after being pried loose with stripping bars are allowed to fall on to two rope slings and eased to the floor to prevent breakage. To make these slings, two 4 in. by 4 in. posts are bored through near the top to take a rope. The rope is knotted at one post and is carried down and fastened around a cleat on the other post. The rope should be long enough to cover one bay between columns. The posts are wedged up tightly between floors alongside the girders, and they should be a little longer than the clear story height. Two slings per bay will be sufficient. (*Fig.* 48.)

Panels should be immediately cleaned of any adhering concrete.

Re-erection.—The size of the columns will usually reduce on the upper floors so that when re-erecting the forms it will be found that the beam and girder forms framing into the columns will be too short because of the longer clear span. The girder bottom is lengthened at each end by half the increase in span by splicing on pieces of plank with cleats ; the joint will be stronger if bevelled, and it must be done each end to keep the beam spacing symmetrical. The sides are similarly lengthened by cleating on a narrow strip at each end. For each reduction in size of column it is better to take off these strips and add wider ones so that there will be only one patch instead of several. On upper floors these patch strips can be used to give the necessary clearance for stripping instead of using the bevelled battens, as they can be taken off before stripping the sides.

The slab panel will also have to be pieced out at the columns. This can be done as for columns by cleating on short pieces, but it makes rather a clumsy joint as the edges of the panel are bevelled where it rests

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on the column, so the new pieces have also to be bevelled to fit or else the bevels are sawn off. A neater joint is made by sawing off each corner of the panel where it comes against the column on the diagonal, so that it will just miss the corner of the column, at the time the panel is made up. The corner is then filled in with one or more boards parallel to the diagonal, cleating on the outside board. At each reduction these boards are removed and new ones put in their place.

If beams reduce in depth on upper floors the sides are simply dropped the required amount (not sawn off at the bottom) and notches are cut for the post caps. If the depth increases it will be an advantage to have had the sides long in the first place, so that only the notches will have to be patched and not the whole form.

When building the forms in the first place the required changes on upper floors should be carefully noted and the forms designed so that the changes can be made with the least expense.

Re-shoring.—In order to maintain a satisfactory speed it will generally be necessary to pour a floor when the floor below has not attained sufficient strength to support the load coming on the shores, so that after stripping the lower floor some or all of the shores must be replaced. These should be put back immediately after stripping and wedged up again under the beam, but not so tight as to strain the concrete. It is best to order sufficient beam bottoms for two full floors and sufficient shores for two and a half floors, so that the second floor can be fully formed and poured before removing the first floor shores—the sides and slab panels can be used on the second floor. The first floor is then stripped and half or two-thirds of the shores replaced; all the bottoms and half the shores are used on the third floor, and so on. In this way there is always one story fully shored and one story half or two-thirds shored below the floor being poured; in addition there should be a few shores in the third story below, say, one at the centre of each beam and girder, which are removed after concreting (*Fig.* 49). With re-shoring a floor can be poured every 8 to 10 days.

With long spans and heavy beams it may be necessary to leave the shores in for a considerable time, in which case to be able to use the timber in the bottoms, at the same time never leaving the beam unsupported, a special design can be made.

Spans of 30 ft. to 35 ft. should have one shore undisturbed during stripping, which can be done as follows (*Fig.* 50): Two saw cuts about 10 in. apart are made right through the beam bottom at the centre, giving a loose piece which is cleated on again during erection. A 6 in. by 6 in. shore is placed in the centre of the loose piece, only instead of the cap being placed on top of the post a short piece of 2 in. plank is placed each side of the post flush with the top, the nails projecting for easy withdrawal. The saw cuts are made at an angle of 45 deg. The remaining shores are placed in the ordinary way. To strip, the two cross pieces and all the shores except the centre one are removed, when the beam box can be lowered and turned around as a whole, or the sides can be stripped first and then the shores and bottom removed.

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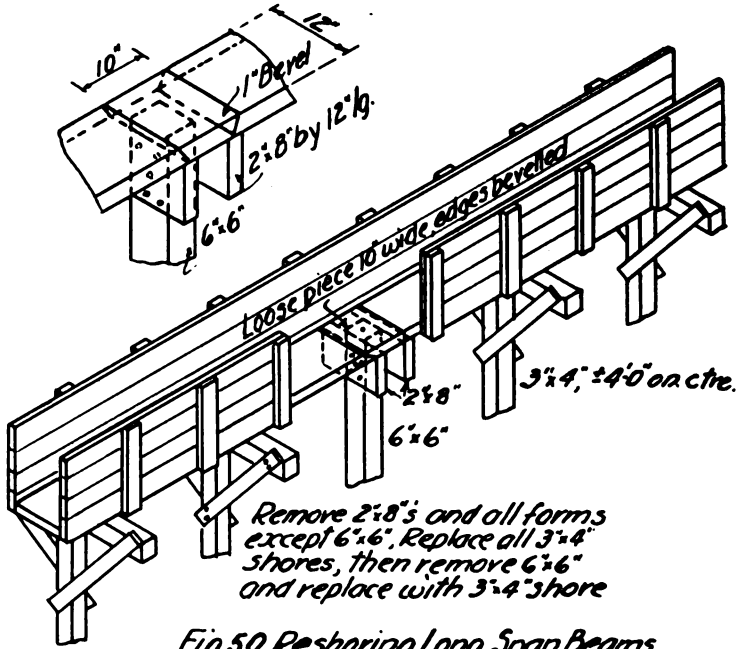

Fig 50. Reshoring Long Span Beams

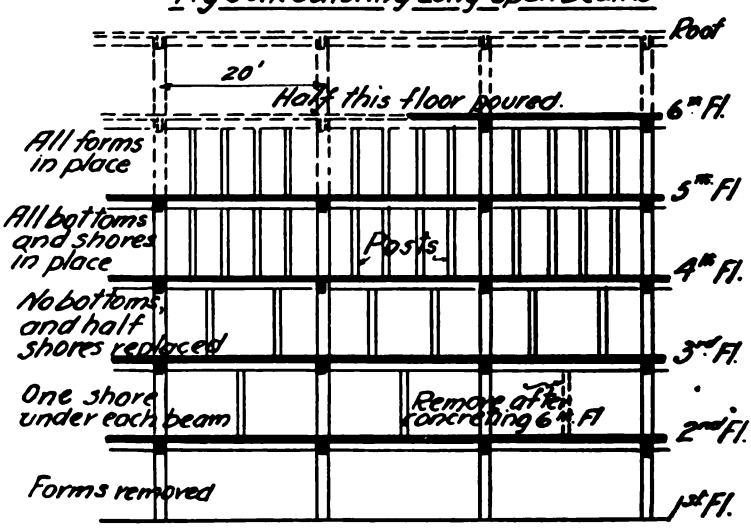

Fig. 49. Reshoring Six Storey Building.

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All the shores are then replaced and the 6 in. by 6 in. can be removed if desired and replaced by a smaller post.

For longer spans still, two permanent shores should be left in the same manner on two loose pieces, and as the sides will be heavy they should be cut in two or three places and cleated together again after erection, so that each part can be removed separately by removing the cleats.

Replaced shores must be cross braced as when first used.

Very High Beams.—When the story height is exceptionally high, say 25 ft. and over, and the beams are heavy, it is inadvisable to use single post shores unless they are made heavy; two post bents are better. These are shown in *Fig.* 46, and consist of two 4 in. by 4 in., or if necessary 6 in. by 6 in., posts about 4 ft. apart, well cross-braced and supporting the beam bottom on two 2 in. planks, one nailed on each side of the posts. They should be deeper than required to carry the load so as to decrease deflection which would cause bending in the posts. Sway bracing should connect the bents longitudinally. If the slab span between the beams is long, ordinarily requiring a centre ledger, it would be better in this case to use a heavier joist sufficient to span from beam to beam and so avoid high shoring under the slab.

Brackets.—If the brackets are not large they can be built as an integral part of the beam form, as in *Fig.* 47. There should be a shore at the start of the bracket, and the joint in the bottom should be on a bevel. If the brackets are large they will be clumsy to handle with the beam form, and are better made as separate units, cutting through the sides and cleating after erection. They can also be built as shown for column brackets.

Very heavy and long brackets should have the shores placed at right angles to the bottom of the bracket, and they must be held from slipping on the floor.

Estimating Cost.

Slabs and beams are usually estimated separately, as the proportion of slab area to beam area may vary considerably. The slab is measured over the whole area, not deducting for beams and girders, since the excess area—about 15 per cent.—will cover the cost of making bevels, fitting around columns, etc. Beams and girders are measured by the surface area in contact below the slab; that is, the area of two sides plus bottom, the length being taken centre to centre of columns or girders.

Slab forms, including joists, will require about $\frac{1}{6}$ cu. ft. of timber per sq. ft. of area. Beam and girder forms, including posts and bracing, will require about $\frac{1}{3}$ cu. ft. of timber per sq. ft. of area in contact. The proportion of beam and girder area to slab area on an average job is about 50 per cent.—rather more than less—and since they take twice as much timber per sq. ft a quick estimate of the amount of timber required can be obtained by figuring $\frac{1}{3}$ cu. ft. over the area of slab and dividing by the number of times the forms are to be used.

For re-shoring, a complete set of shores and braces will require about

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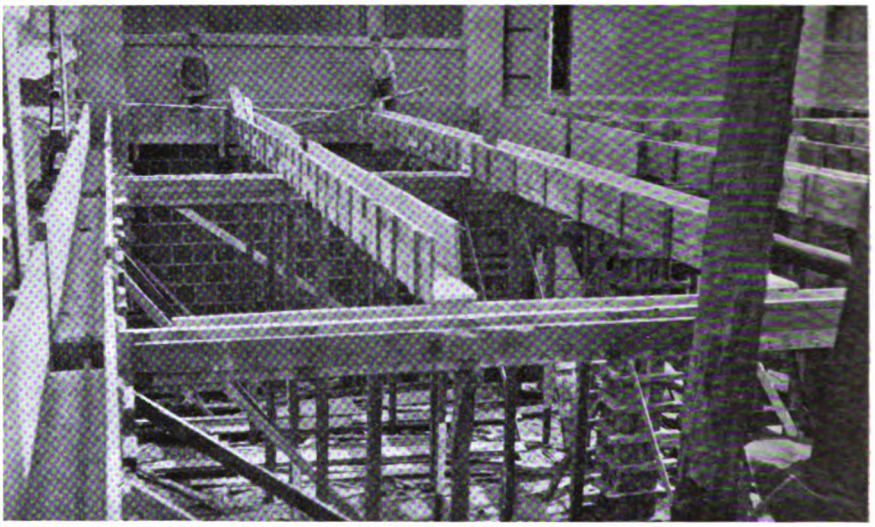

FIG. 50.—GIRDER SIDES BUILT TO UNDERSIDE BEAM BOTTOMS AND COMPLETED WITH PANELS FROM BEAM TO BEAM.

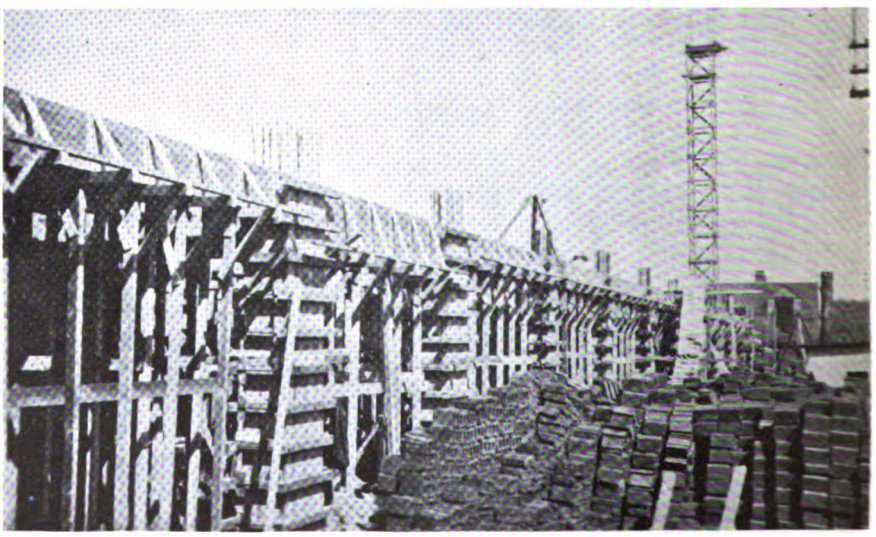

FIG. 51.—SUPPORTING EXTERIOR BEAMS DIAGONALLY FROM SHORES.

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0·15 cu. ft. of timber per sq. ft. of area of beam, or the number of shores required can be estimated at so much each.

About 10 per cent. should be added to the timber quantities for each time the forms are reused, to allow for repairs, etc.; this is a liberal allowance. Brackets are figured in with beams.

To make up 100 sq. ft. of slab forms, including joists, will require about $2\frac{1}{2}$ carpenters' time plus 1 hour labourers' time. Erection in place will require about $1\frac{1}{2}$ hours carpenters' time plus $1\frac{1}{2}$ hours labourers' time, and stripping about $\frac{1}{2}$ hour carpenters' time plus 2 hours labourers' time.

No difference is made whether the joists are loose or attached to the panels; in the former case assembly will cost less and erection more, but the total cost will be about the same, though fixed joists should work out a little cheaper.

Cleaning and hoisting will each require about 1 hour labourers' time.

Cost of 100 sq. ft. of slab forms, first erection :—

$4\frac{1}{2}$ hrs. carpenter at..... =
$4\frac{1}{2}$,, labourer at ..... =
timber 100/6, say 17 cu ft. at.. =

For each subsequent erection cost will be :—

2 hrs. carpenter at..... =
$5\frac{1}{2}$,, labourer at..... =
timber 10 per cent. of 17 = 1·7 cu. ft. at..... =

Cost of 100 sq. ft. of slab forms, used 4 times, calculated on whole area:

$2\frac{5}{8}$ hrs. carpenter at..... =
$5\frac{1}{2}$,, labourer at..... =
timber $\frac{17 + 5{\cdot}1}{4}$ say $5\frac{1}{2}$ cu. ft. at =

Labour on beams and girders is figured at an average cost, although girders will actually cost a little more than beams.

To make up 100 sq. ft. of beam and girder forms will require about 4 hours carpenters' time plus 2 hours labourers' time. Erection will require about 5 hours carpenters' time and 5 hours labourers' time, and stripping about $1\frac{1}{2}$ hours carpenters' time plus 3 hours labourers' time. Cleaning and hoisting will each require about 1 hour labourers' time.

Cost of 100 sq. ft. of beam and girder forms, first erection :—

$10\frac{1}{2}$ hrs. carpenter at..... =
10 ,, labourer at..... =
timber 100/3 = $33\frac{1}{3}$ cu. ft. at.. =

Each subsequent erection will cost :—

$6\frac{1}{2}$ hrs. carpenter at..... =
10 ,, labourer at..... =
timber 10 per cent. of $33\frac{1}{3}$ say $3\frac{1}{3}$ cu. ft. at..... =

Cost of 100 sq. ft. of beam and girder forms used 4 times :—

$7\frac{1}{2}$ hrs. carpenter at..... =
$10\frac{1}{4}$,, labourer at..... =
timber 10·83 cu. ft. at..... =

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Both for slabs and beams 1 hour labourers' time is added for lowering the timber from the fourth floor to the ground, but no allowance is made for cleaning it again as some of it will be worthless, some may be sold as it is, or the cleaning may be charged to the next job.

For replacing and stripping shores about ¼ hour carpenter and ¼ hour labourer should be allowed additional for each shore. Additional labour should be allowed for brackets, from 1 to 4 hours carpenters' time apiece, depending on the size.

Bevel strips on beam edges should be estimated on the total number of lineal feet required. A carpenter will place 50–60 lineal feet per hr.

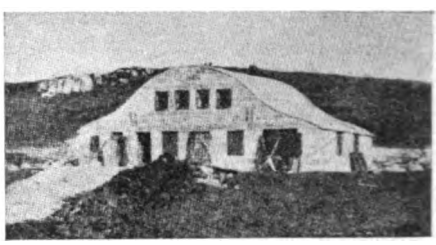
These costs are for ordinary conditions; extra high beams and unusual features will need additional labour charges.

(*To be continued.*)

Roofs for Exposed Positions.

In the course of some interesting notes in the "Journal of the Institution of Municipal and County Engineers," Mr. C. H. Cooper, late Borough Engineer of Wimbledon, describes a type of roof he has devised to withstand the gales of the Scilly Isles which sometimes reach over 100 miles per hour, and which at the same time would not be too costly, would prevent rain entering no matter how fierce the gale, and withstand the corrosive action of sea-water.

The design decided on is shown in the illustration. The curves on each side are got by three arcs of 18-ft. radii. The roof is laid in concrete 3-in. thick, composed of 4 parts coarse sea sand to 1 part of Portland cement reinforced with wire netting with a 4-in. mesh and five-sixteenth-inch steel rods placed 3 ft. apart. Two coats of cement wash were applied to the outside while the concrete was still green. The roof was completed in December, 1923, and the writer states: "Subsequent gales have in no way affected it. Except where a settlement of the foundations has occurred it has remained perfectly weathertight, and the crack caused by the settlement has been made weatherproof by cement grout." Smaller buildings have been similarly roofed with concrete only 2 in. thick, and these have also remained perfectly weatherproof. The writer has found no difficulty in making concrete 2 in. or 3 in. thick waterproof, provided the surface is trowelled and floated before it sets hard:



Roof designed to resist Gales.

If the concrete is good, with a rich mixture of cement, sufficient working of the weather face will ensure it being watertight. The walls and floors of these buildings are also of concrete, and the writer states that some concrete floors 2 in. thick only have been covered with rain water for months without any sign of damp appearing on the underside.

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The Foreman's Guide to Concrete.—XI.

A MANUAL FOR ALL ENGAGED IN CARRYING OUT CONCRETE AND REINFORCED CONCRETE STRUCTURES.

By Albert Lakeman, M.S.A., M.I.Struct.E.

BASES & FOUNDATIONS.

THE foundations of a building form the means of transmitting the whole of the loads carried by the structure, including its own weight, to the soil beneath, and the importance of well-designed and properly-executed foundations will be realised. Owing to the monolithic nature of a reinforced concrete building a slight settlement at one point is not likely to cause disaster, as the danger of collapse is minimised when compared with the older types of construction, and in this respect there is a greater factor of safety due to the perfect connection between the various members enabling one portion to assist another, but this does not mean that any neglect is permissible when dealing with foundation work. Settlement will always take place with new work, but in a well-designed building this is so minute that it is not perceptible, and provided the settlement is uniform over the whole of the work no harm is caused. But if the settlement is excessive, and especially if the settlement under one portion is greater than under another, there is a risk of columns and walls moving out of the vertical line. Stresses will then be set up for which no provision has been made, cracks will occur, and the strength of the building will be affected.

There are many points which have to be considered when designing foundations, and among the principal ones are the following :—

(1) The safe resistance of the soil,

(2) The atmospheric influence on the soil,

(3) The size, shape, and strength of the foundation to give proper distribution, and

(4) The possibility of the withdrawal of water or soil during future work in the vicinity.

Safe Resistance of Soil

The safe resistance of the soil is the first factor to be considered because this will determine the area over which the load must be distributed, and the object of the foundation will be to spread this load over an area sufficiently large to prevent a pressure at any point which is greater than the weight which the soil can safely bear. In the regulations of the L.C.C. it states that the pressure of foundations on the natural ground shall not exceed the following :—

Natural bed of soft clay or wet or loose sand—1 ton per sq. ft.

Natural bed of ordinary clay or confined sand—2 tons per sq. ft.

Natural bed of compact gravel, London blue clay or chalk—4 tons per sq. ft.

As an example, if we assume a reinforced concrete column carrying a load of 100 tons, the area of the base must be sufficient to spread this load to keep the pressure per sq. ft. within the limits given above ; if the foundation is in soft clay the base must be 100 sq. ft. in area, or, say, 10 ft. square ; for ordinary clay 50 sq. ft. is necessary or a base just over 7 ft. square ; and for compact gravel an area of 25 sq. ft. is required, which will be given by a base 5 ft. by 5 ft.

It will be seen that the nature of the soil is therefore the most important point to be considered, because it is impossible to commence the design of the foundations until this is known and the area is calculated, as this is the first step in arriving at the amount of material required. The area necessary determines the spread outward from the weight-carrying member, or, in other words, the projection of the base, and this projection has to be calculated according to its length and the pressure per square foot.

Influence of Atmosphere on Soil

The second point is the influence of the atmosphere on the soil, and while this is not so important as the safe resistance of the soil it is an item that must be considered. The weather conditions and the atmosphere generally affect the soil below the surface as well as the surface of the ground itself, and if the foundations are not carried down below the level of the

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atmospheric influence they will be subject to varying conditions which cause movement and possible settlement.

The level of the limit of atmospheric influence in this country is generally reckoned at about 3 ft. from the surface, and this is therefore the minimum depth to which the foundation should be taken when dealing with work of importance. The action of the atmosphere is liable to be more pronounced in some soils than in others, and the designer will be influenced by the conditions. In a clay soil, for example, the atmosphere in the summer will usually cause the soil to contract considerably, and cracks will occur on the surface and extend down to the level of the atmospheric influence, while the atmospheric conditions in the winter will cause the clay to swell again; the soil is therefore constantly on the move. If the foundations are constructed within the soil which is subject to these changes it will be obvious that the foundations will also be subject to movement, and in many districts where clay soil abounds cracks due to this cause alone can be seen in the walls of the buildings.

Frost also causes moisture in the soil to expand, and when a thaw sets in contraction will follow; this means a movement down to the level of the limit of the atmospheric influence. The action of the expansion of the soil during frost is well illustrated in the case of a road surfaced with slag or water-bound macadam, as when a thaw occurs the surface will be found to have become broken up and generally disintegrated owing to expansion causing the particles to separate.

The importance of this feature in foundation work will depend so much on circumstances that no general rule can be given other than the taking down of the bases of all main members so that the underside is below any possible movement of the soil during changes of atmosphere.

Distribution of Load

The third point for consideration is the size, shape, and strength of the foundation to give proper distribution, and it is here that the designer has to make detailed calculations and develop the most efficient and economical type to suit the loads and conditions. Although the foundations are the first portion of a building to be executed on the site they are the last item to be dealt with by the designer. The sizes and weights of the whole of the members in the structure must be determined before the total loads on the foundations can be found, and for this reason the designer has to work downward from the roof and collect the loads on the walls and columns as each floor is reached until the total is obtained at the lowest point. When the total load on a column has been calculated and the safe pressure on the soil is known the design of the base can be dealt with.

The necessity for a properly-designed base can be illustrated by a few simple diagrams and notes. In *Fig.* 71 a reinforced concrete column is indicated, and if we assume that this carries a load of, say, 100 tons, and it has a sectional area equal to 3 sq. ft., it would mean a pressure of over 33 tons per sq. ft. on the soil below the column at the point A if the member were simply taken down into the ground without any special base. The maximum pressure per sq. ft. with firm gravel is only 4 tons, and it is therefore obvious that a pressure of over 33 tons would cause the soil to be compressed by the excessive weight and that the column

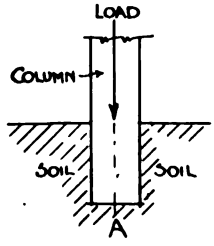

FIG. 71

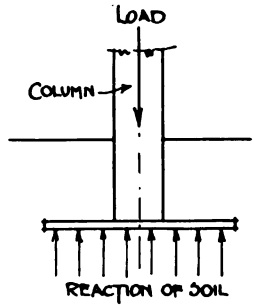

FIG. 72.

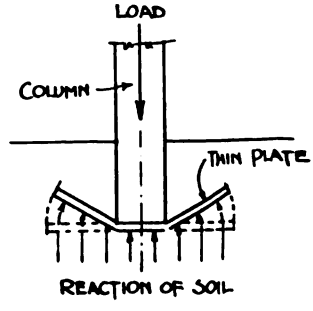

FIG. 73

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when fully loaded would sink into the ground. In order to overcome this, some means of spreading the pressure over a larger area must be found, and we can put a plate under the column as shown in *Fig.* 72. With a load of 100 tons and a safe resistance of 4 tons per sq. ft. the size of this plate must be equal to 25 sq. ft., or 5 ft. by 5 ft. It will be clear that the column cannot sink into the ground without pressing the plate down also, and if the size of the plate keeps the pressure within the safe limits of the soil then the latter will not give way and the plate will be kept in position.

This arrangement will thus deal with the question of distribution provided the plate is strong enough to transmit the load from the column to the soil evenly over the whole area of 25 sq. ft., but if the plate is not sufficiently stiff and is composed, for example, of a thin metal sheet it would be bent upwards where it projected beyond the column, as illustrated in *Fig.* 73. This shows that the portion of the base which projects from the column must be calculated as a cantilever that carries a distributed load equal to the pressure on the soil, because the pressure must be met by the reaction of the soil, and the base is the medium through which the concentrated pressure on the column is neutralised by the distributed reaction of the soil.

If the diagram in *Fig.* 73 is turned upside down, the cantilever effect will be realised, and there should be no difficulty in understanding the principles which are here briefly explained. The actual calculations are not quite so simple as might appear at first sight because the base is a square on plan and the projection on each of the four sides consists of part of a triangle, as indicated by the shaded portion in *Fig.* 74. The whole of the pressure

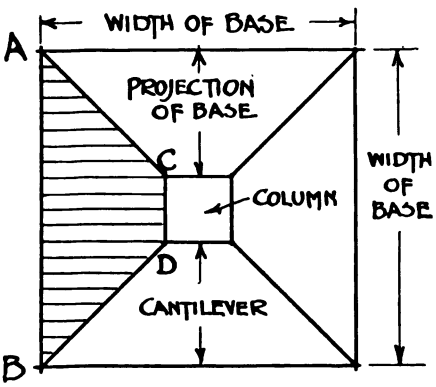

FIG. 74.

on this shaded area, marked ABCD, has to be transmitted through the material on the line CD, and it is therefore necessary to calculate the pressure and the centre of this pressure—or, in other words, the point at which it can be considered as acting—before the bending moment at CD can be found. The total pressure on ABCD multiplied by the distance from the centre of the pressure to the line CD will give the maximum bending moment

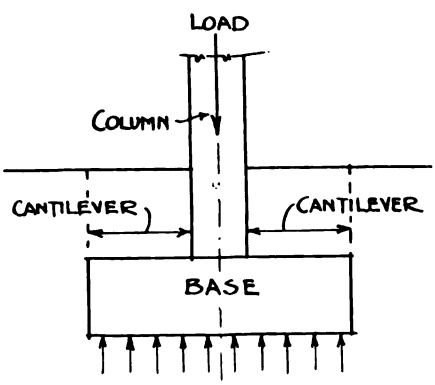

FIG 75

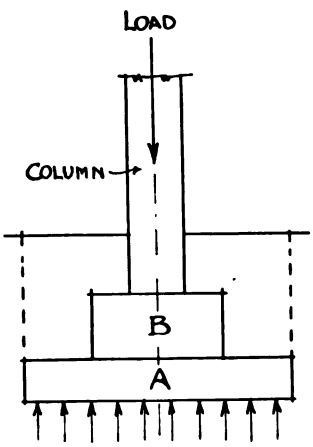

FIG 76

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at this point, and from this bending moment the necessary resistance of the steel and concrete can be calculated. It is not proposed to carry the notes into the subject of the actual calculations to be made, as we are dealing with the general principles only.

The most simple type of base is that indicated in *Fig.* 75 where the projections consist of a thick layer of reinforced concrete of the necessary area to give the requisite distribution. This type is, however, not usually economical because the thickness of the base at the junction with the column, which is necessary to resist the bending moment at this point, is excessive at the outer edges, and consequently more concrete is used than is theoretically necessary.

In order to overcome this waste the foundations are sometimes stepped, as shown in *Fig.* 76, where the thick concrete at the junction is reduced at the outer edges. In this type the excavation is usually made as shown by dotted lines, and the lower concrete is put in to the required thickness for the full size of the excavation. A simple form is then placed on this lower concrete (marked A) in order to allow the portion marked B to be poured, and above this the actual column is commenced. This type is more economical than that shown in *Fig.* 75 except for very small columns carrying light loads, because in the latter case only a small base is required and the cost of the formwork would not be justified by any appreciable saving in the concrete.

As the maximum bending moment in the base will occur at the line of the junction with the column, and this moment will diminish to the outer edge where it will be nil, the ideal base from the theoretical point of view is that illustrated in *Fig.* 77, where the concrete is calculated according to the variation in the bending moment with the result that the top of the base is splayed from the junction with the column to the outer edges, where it is a few inches thick only. This base is constructed without any waste of concrete and when executed it is perfectly satisfactory, but it is not always economical in initial outlay owing to the difficulty with the formwork. These forms are not simple to make and they require accurate and careful workmanship both in making and fixing, but the chief disadvantage arises from the fact that they must be anchored down or loaded during the placing of the wet concrete to prevent lifting by the pressure of the wet material. This pressure is considerable in large and deep bases, and the necessity for keeping the forms down involves the use of labour which could be used elsewhere and also causes some delay in the execution of the work. This type of base

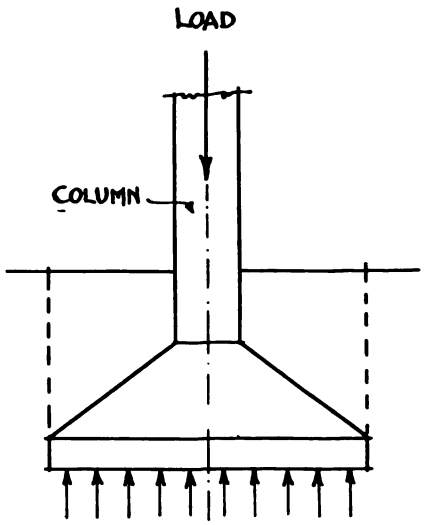

FIG. 77.

is, however, very generally used, as it is the shape which will naturally appeal to the designer of a scientific material where waste is not permissible.

Changes in Soil

The last point for consideration is the possibility of the withdrawal of water or soil during future work in the vicinity of the foundations, and the importance of this will depend entirely on the circumstances of each particular case. It will be clear, however, that if the foundations are designed to give a certain pressure on the soil, and at a later date this soil is influenced by adjacent excavations which cause water and fine particles to be drawn away, then the soil will probably contract, and the necessary resistance will not be given. This will cause settlements in the structure, and as the settlements will probably be unequal cracks and defects will occur. This point is men-

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tioned as it explains why foundations are sometimes indicated on the drawings at a lower level than that required to take them down below the line of atmospheric influence or on to a firm soil; as the designer will make provision for future work where it is possible to ascertain the requirements.

Before leaving the subject of column bases a few notes on the arrangement of the steel may be useful. The amount and disposition of the steel will, of course, depend on the load to be carried, and the general arrangement will also depend on

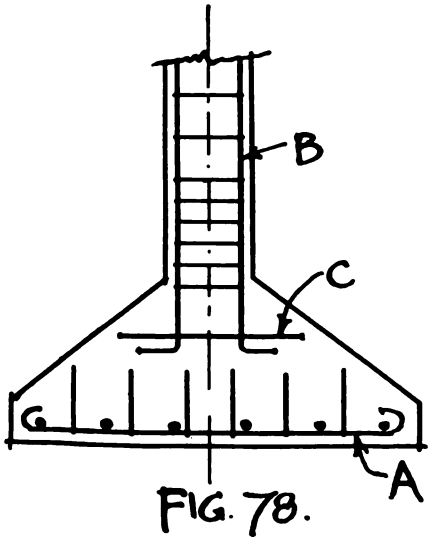

FIG. 78.

the method adopted by the designer and the particular system or type which is used. The outline of a typical foundation base is given in *Fig.* 78, and it will be noticed that the steel is lettered A, B and C. The rods marked A indicate the main tension steel in the underside of the base and these will run across the foundation in both directions to give the resistance to tension caused by the bending moment in each cantilever portion. The steel marked B indicates the main vertical reinforcement in the column itself which is carried down into the base. These rods are bent outwards at the ends to give a right-angle turn, and this is done to prevent the end of the rod pressing directly on to a small area of concrete and causing excessive compression.

As already mentioned the stress on the steel will be 15 times as much as that on the concrete in compression, and it will be obvious therefore that if the end of the rod bears directly on to the concrete without any provision for distribution the load on the rod will be passed to the concrete on a small area, and there will be a serious risk of the rod pushing downward through the base. The steel marked C is provided to give resistance to shear as there is a tendency for the

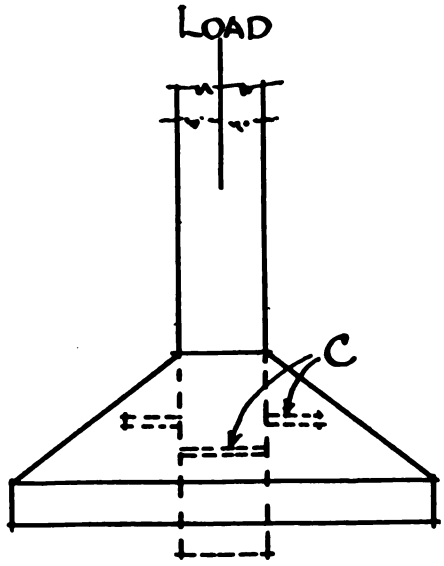

FIG. 79.

column itself to shear or push its way through the base, as indicated by the dotted lines in *Fig.* 79. This tendency is the same as that illustrated in connection with vertical shear in the notes on stress, and where the resistance to this shear is not sufficient without special provision some steel rods are placed in the base as shown. Upon reference to *Fig.* 79, it will be seen that the column could not shear through as indicated by the dotted lines without shearing through the steel marked C. The base will also require stirrups for the anchorage of the main tension steel and to resist horizontal shear, and the ends of the tension steel must be hooked as previously explained. In some foundation bases the steel pro-

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vided is somewhat complicated, and considerable care is necessary to ensure the work being carried out accurately.

If the foreman should experience any difficulty at the outset it is advisable to assemble the steel for one base as a sample, on the surface of the ground, and if necessary obtain the approval of the designer before placing any reinforcement below ground level, especially if there is any doubt regarding the correct interpretration of the drawings.

The foundations for walls as distinct from columns will depend entirely on circumstances, but generally speaking the walls in a reinforced concrete structure are not usually designed to support heavy loads, and the foundations are therefore of minor importance as the weights are transmitted to the columns and piers. In the case of retaining walls the foundations are designed in conjunction with the vertical portion of the wall, as previously mentioned; and these do not come under the ordinary heading of bases or foundations.

When it is necessary to erect a reinforced concrete structure on "made" ground (i.e. where the ground has been filled up by tipping soil and rubbish to give a level above the natural ground level) it is important to design the foundations with due regard to the conditions of the site, because "made" ground will usually have a poor bearing capacity and it is also liable to sink under the action of the weather. Unless "made" ground is exceptionally deep it is advisable to carry all the weight-carrying foundations down to the solid natural soil, but in some cases where the "made" ground is well consolidated and the structure is not of a heavy type the loads can be distributed by means of a raft foundation. This raft will consist of a large reinforced concrete slab constructed over the whole area of the building, and having reinforced concrete beams or ribs at intervals to divide the slab into panels. By this method the loads will be distributed over a large area and one portion of the building cannot settle without the raft breaking or the whole building and raft going down together. If the raft is properly designed to distribute the loads and executed in accordance with the drawings there is no fear of a break taking place, and if the whole raft and building settle down slightly over the whole area in a uniform manner then no harm will be done. It is sometimes necessary to use reinforced concrete piles under the raft to assist in the resistance to settlement, and special care and experience are required to carry out this class of work.

A great deal could be written about foundations and bases as these features of constructional work form a subject of considerable importance, but sufficient has been given to indicate the main principles that govern the design of ordinary types. There is no part of a structure which calls for more supervision and care on the part of the foreman, and it may be said that if the foundations are well and truly executed the scheme is well on the way to success.

(*Concluded.*)

Book Review.

The Strength of Materials. By Ewart S. Andrews, B.Sc.Eng., A.M.Inst.C.E.
[London: Chapman & Hall, Ltd. Pp. 608, 264 illus. Price 13s. 6d. net. Second Edition.

THIS is a second edition of Mr. Ewart Andrews' well-known work dealing with the design of structures and machines from the standpoint of strength. The subject is dealt with in a thoroughly practical manner, and its value is increased by the numerous worked examples and exercises which are given. In this new edition the author has not altered the treatment of the subject, the only variation from the first edition being the addition of the properties of the new British Standard Sections. We have no doubt a third edition of this popular work will soon be necessary.

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Thames ballast	,,	10	6
Broken brick (¾ in.)	,,	11	6
Best British Portland Cement	per ton 58s. to	63	6
"Ferrocrete" Rapid-Hardening Portland Cement	delivered London 10s. per ton extra		
"Super-Cement"	per ton 88s. to	93	0
"Lightning" Brand Aluminous Cement	,, 108s. to	113	6
Ciment Fondu		130/- ex stores	

BOARDING FOR SHUTTERING—		Sawn. s.	Sawn. d.	Wrot. s.	Wrot. d.
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—	
3 in. by 4 in.	from £23 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £23 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅝ in. to 2¾ in. Rounds	per cwt.	13	0
3/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds	,,	14	6
¼ in. Rounds	,,	15	6

Breeze Slabs per yd. super: 2 in., 1/11; 2½ in., 2/4; 3 in., 2/9; 4 in., 3/6.

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 : 2 : 4—			s.	d.
Do.	do.	in foundation	per yard cube	48	6
Do.	do.	in columns	,, ,,	53	6
Do.	do.	in beams	,, ,,	53	6
Do.	do.	in floor slabs 4 in. thick	per yard super	5	8
Do.	do.	in floor slabs 6 in. thick	,, ,,	8	5
Do.	do.	in floor slabs 9 in. thick	,, ,,	12	3
Do.	do.	in walls 6 in. thick	,, ,,	8	8

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—		s.	d.
From ¼ in. to ⅝ in.	per cwt.	25	0
,, 3/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2¾ in.	,,	23	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1¼*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2¼*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.* (per bend per cwt.).

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.* (per bend per cwt.).

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	,, ,,	1	1½
Do. do. to sides and soffits of beams, average 9 in. by 12 in.	,, ,,	1	1½
Do. do. as last, in narrow widths	,, ,,	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Carpenters working on old shuttering, 1/10½; Labourers on building works, 1/4½; Men on mixers and hoists, 1/5½; Bar-benders, 1/5½.

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Prospective New Concrete Work.

ABERDARE.—*Road.*—The U.D.C. has decided to build a roadway in the bed of the canal.

ALSAGER.—*Filters.*—The U.D.C. has decided to proceed with a water filtration scheme.

ARNOLD.—*Houses.*—The borrowing of £5,500 for the erection of 12 houses by the U.D.C. has been sanctioned by the Minister of Health.

AYR.—*Filters.*—It is proposed to make additions and alterations at the filter plant at Knockgarden, at a cost of £16,650. The work is to be undertaken by the Ayr Corporation.

BELSTONE.—*Reservoir*—The Okehampton R.D.C. proposes to erect a reservoir, estimated to cost £300.

BLACKBURN.—*Houses.*—The T.C. is to build 160 houses on the Brownhill site.

BLACKPOOL.—*Houses.*—The Corporation proposes to erect 130 houses.

BLACKWATER.—*Bridge.*—A bridge is to be built over the canal at Blackwater by the Hampshire C.C., at a cost of £51,000.

BLAIRGOWRIE.—*Sewage Works.*—It has been decided by the T.C. to borrow £3,000 for sewage works.

BLYTH.—*Bathing Pool.*—At a cost of £7,000, a bathing pool is to be built at Blyth to the design of the Borough Surveyor.

BRAMLEY.—*Houses.*—The Dalton Main Colliery Co., Ltd., of Haunderwell, is to build 262 houses at Bramley for housing the workers at the colliery.

BURNHAM-ON-SEA.—*Water Supply.*—The U.D.C. has decided to carry out improvements in connection with the water supply works, and has appointed Messrs. Wilcox and Raikes consulting engineers for the works.

CARDIGAN. — *Coast Defence.* — The County Surveyor of Cardigan is preparing a scheme for coast defence at Aperporth, including the construction of a concrete wall.

CHESTER.—*Houses.*—A loan of £49,104 has been applied for by the T.C. for the construction of 102 houses at Rock Ferry.

CHESTERFIELD.—*Houses.*—The R.D.C. proposes to build 100 houses in the northern area of its district, and has approved a site at North Wingfield for a further 200 houses.

CHESTERFIELD.—*Sewage Works.*—The T.C. has approved the expenditure of £90,900 for the provision of a sewerage scheme at Brimington, Sutton and Staveley.

COLCHESTER.—*Concrete Road.*—The Corporation's unemployment relief scheme includes the construction of an 18-ft.-wide reinforced concrete carriageway, estimated to cost £12,000.

(*Continued on page* 533.)

Concrete Memoranda.

IN response to several requests from readers we give below the weights of various materials used in reinforced concrete, and propose to make this a regular feature complementary to the Data on the preceding page. The figures in all cases are average figures, and may vary in different cases.

Cement.—1 cu. ft. of Portland cement is calculated as weighing 90 lbs.; when filled very loose it may weigh as little as 75 lbs. per cu. ft.; if packed tight it will weigh 110 lbs. per cu. ft.

Water.—1 cu. ft. of water weighs 62½ lbs.; 1 gallon = 10 lbs.

Sand and Aggregate.—1 ton = 21 cu. ft. river sand; 22 cu. ft. pit sand; 22 cu. ft. ballast; 23 cu. ft. coarse gravel; 24 cu. ft. clean shingle; 40–50 cu. ft. broken brick.

Reinforcement.—Weight of square and round rods per ft.: ¼-in. sq. rods = 0·22 lb.; ¼-in. dia. round rods = 0·17 lb.; ½-in. sq. = 0·86 lb.; ½-in. round = 0·67 lb.; ¾-in. sq. = 1·92 lb.; ¾-in. round = 1·51 lb.; 1-in. sq. = 3·42 lbs.; 1-in. round = 2·62 lbs.; 1¼-in. sq. = 5·34 lbs.; 1¼-in. round = 4·19 lbs.; 1½-in. sq. = 7·69 lbs.; 1½-in. round = 6·04 lbs.

Concrete.—Weight of 1 : 2 : 4 concrete per cu. ft.: with coke breeze aggregate, 100 lbs.; with broken brick aggregate, 125–130 lbs.; with limestone aggregate, 135 lbs.; with shingle aggregate, 145 lbs.

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CONISBROUGH.—*Houses.*—The U.D.C. is proposing to erect 60 houses by direct labour.

COOKSTOWN.—*Houses.*—Application has been made by the R.D.C. for sanction to a loan for the erection of 99 houses.

CRUMLIN.—*Houses.*—The Abertillery U.D.C. proposes to build 100 houses on the Suffryd Estate.

DARLINGTON.—*Houses.*—The T.C. has definitely decided to proceed with the erection of 100 houses.

FALKIRK.—*Concrete Bridge.*—The Stirlingshire C.C. proposes to construct a reinforced concrete bridge over the Carron at Larbert.

FELLING.—*Houses.*—Sanction to a loan of £34,000 is to be applied for by the U.D.C. for housing purposes.

GELLYGAER.—*Bridge.*—A bridge is to be constructed over the Rhymney at Fleur-de-Lis.

HEMSWORTH.—*Houses.*—The R.D.C. has decided to build 250 houses at South Kirby.

HENDON. — *Bridge Widening.* — The L.M.S. railway bridge at Hendon is to be widened.

HITCHIN.—*Houses.*—The U.D.C. proposes to build 500 houses within the next two years.

HULL.—*Bridge.*—It has been decided to seek powers for the erection of a new North Bridge at Hull at a cost of about £275,000.

IRTHLINGBOROUGH. — *Bridges.* — The Northamptonshire C.C. proposes to spend £50,000 on the construction of bridges at Irthlingborough.

LAMPETER.—*Bridge Widening.*—The Cardigan County Council is considering the widening of Lampeter Bridge, and has instructed the Surveyor to prepare schemes for the work.

LANARKSHIRE.—*Houses.*—An application to the Scottish Board of Health has been made by the Lanarkshire District Committee for sanction to the erection of 1,000 houses.

LARNE.—*Road.*—The Antrim C.C. has applied for sanction to construct a road from Larne to Mounthill.

LONGBENTON.—*Houses.*—The U.D.C. has applied for permission to erect 56 houses at Forest Hall.

LONGFORD.—*Bridge Widening.*—The bridge over the river at Longford is to be widened by the Warwickshire County Council.

MANCHESTER.—*Houses.*—The Corporation has been recommended to apply for a loan of £359,055 for housing purposes at Moston.

MANCHESTER.—*Sewage Disposal.*—The T.C. proposes to spend £456,000 for sewage disposal works.

MARPLE.—*Sewage Works.*—Extensions to the sewage disposal works at Marple are to be carried out by the U.D.C., and land has been purchased for the purpose.

MILNGAVIE.—*Bridges.*—The Burgh Surveyor is preparing plans for the widening of the bridge over the Allander water.

NEWCASTLE-ON-TYNE. — *Road.* — The Corporation proposes to spend £750,000 on the construction of a road from Pilgrim Street to Barras Bridge.

NEWPORT.—*Bridge.*—The scheme for the reconstruction of Newport Bridge is estimated to cost £170,000.

NEWPORT (Mon.).—*Bridge.*—The T.C. is considering the erection of a footbridge over the river Ebbw.

NORTH WALSHAM.—*Sewage Works.*—The U.D.C. has applied to the Minister of Health for sanction to borrow £27,500 for sewage disposal works.

NORTHWICH.—*Bridge.*—The widening of the bridge over the Cheshire Lines Railway at Northwich station is under consideration by the U.D.C.

NOTTINGHAMSHIRE. — *Bridges.* — The Nottinghamshire C.C. is proposing to erect bridges in place of the existing level crossings on the Great North Road, at an estimated cost of £50,000.

PORTSMOUTH.—*Houses.*—The T.C. is negotiating for a site in connection with the building of 76 houses.

PRESTON.—*Houses.*—The T.C. proposes to erect 100 houses on the Delaware Street site.

RAWMARSH.—*Houses.*—The U.D.C. has applied for a loan of £47,307 for the construction of 100 houses.

SEAHAM HARBOUR. — *Houses.* — The U.D.C. has decided to invite tenders for the erection of 56 houses.

SEDGEFIELD. — *Houses.* — The R.D.C. has been recommended to apply for permission to erect 300 houses.

SHEFFIELD.—*Houses.*—The sum of £157,475 is to be spent by the T.C. on building 340 houses at Wyborn.

SKEGBY.—*Sewage Disposal Works.*—The Skegby (Yorks) R.D.C. proposes applying for sanction to a loan of £17,000

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for the construction of sewage disposal works at Blidworth.

STONE.—*Sewage Works.*—In connection with its sewage works, the U.D.C. proposes to construct a humus separator and a syphon under the canal, at a cost of £1,280.

STROUD.—*Reservoir.*—A scheme for the construction of a reservoir, estimated to cost £10,000 is being considered by the U.D.C.

SUDBURY.—*Bridge.*—A new bridge is to be built over the District Railway at Sudbury, at a cost of £12,072.

SUNDERLAND.—*Quay.*—The T.C. is considering the construction of a deep-water quay.

TETBURY.—*Reservoir.*—The U.D.C. has applied for sanction to construct a 500,000-gallon reservoir.

TIVERTON.—*Houses.*—The R.D.C. has a scheme in hand for the erection of 104 houses.

TREDEGAR.—*Houses.*—Sanction is being applied for for the erection of 100 houses.

TRERDEL.—*Bridge.*—A scheme for improving and widening Trerdel Bridge is under the consideration of the Cardiganshire C.C.

WALLASEY.—*Houses.*—It is proposed to build 32 houses on the Belvedere Road site, and sanction to the necessary loan is to be applied for.

WEYMOUTH.—*Bridge.*—The T.C. has applied to the Dorset C.C. for a contribution towards the cost of erecting a bridge across the harbour.

WILLESDEN.—*Houses.*—Approval has been received by the U.D.C. to the erection of 67 houses on the Brentfield Estate.

WOOLWICH.—*Houses.*—The Woolwich B.C. has applied for sanction to a loan for the erection of 100 houses on the Eltham Estate.

YEADON.—*Houses.*—Approval has been received by the U.D.C. for the erection of 72 houses at Hawthorn Crescent.

YORK. — *Improvements.* — Among the relief schemes proposed by the T.C. are the culverting of Lang Hall Beck, at a cost of £1,500; concrete surfacing various streets at a cost of over £4,000; and building an arterial road at a cost of about £80,000.

Tenders Accepted.

BAKEWELL.—*Bridge.*—The R.D.C. has accepted the tender of the British Reinforced Concrete Engineering Co., Ltd., at £400, for the construction of a reinforced concrete bridge.

BLACKBURN.—*Bridge.*—The tender of Messrs. Gray's Ferro-Concrete Co., of Glasgow, has been accepted by the T.C. for the construction of a reinforced concrete bridge on the Hennebique system over the L.M.S. Railway at Brownhill.

BOURNEMOUTH.—*Tank.*—The T.C. has accepted the tender of the Premier Cooler & Engineering Co., at £2,005, for the construction of reinforced concrete tanks at the Generating Station.

BRIGHTON.—*Concrete Houses.*—The T.C. has accepted the tender of Messrs. John Laing & Son, Ltd., at £7,648, for the erection of 16 concrete houses.

CAMBRIDGE.—*Houses.*—The T.C. has accepted the tender of Mr. D. G. Underwood, of Brundall, near Norwich, at £21,420, for the construction of 42 concrete houses.

CARLISLE.—*Kerb and Channel.*—The tender of the Cumberland Reinforced Concrete Co., of Harrington, at £575 6*s.* 6*d.* for the supply of concrete kerbing and channels, has been accepted.

CHESTERFIELD.—*Bridge.*—The R.D.C. has accepted the tender of the Reinforced Concrete Construction Co., at £695, for the construction of a bridge at Crow Hole, Barlow.

CHIRK (SALOP).—*Bridge.*—The Salop C.C. has accepted the tender of the Reinforced Concrete Construction Co., at £6,543, for the reconstruction of Chirk Bridge in reinforced concrete.

CLEOBURY MORTIMER.—*Bridge.*—The R.D.C. has accepted the tender of the Reinforced Concrete Construction Co., at £1,761, for the construction of a bridge at Wetton Ford.

FAREHAM.—*Reservoir.*—The U.D.C. has accepted the tender of Messrs. J. Hunt, Ltd., Gosport, at £3,135, for the construction of a 300,000-gallon reinforced concrete reservoir and a 60,000-gallon reinforced concrete water tank.

GOLCAR.—The U.D.C. has accepted the tender of Mr. A. Graham, at £472, for the construction of concrete floors and foundations in connection with the erection of a refuse disposal plant.

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HULL.—*Bridge.*—The Corporation has accepted the tender of Messrs. Langton & Son, Ltd., at £3,948 18*s.* 2*d.* for the construction of a reinforced concrete bridge across the Holderness drain at Preston Road.

ILKESTON.—*Reservoir.*—The T.C. has accepted the tender of Messrs. Mathews and Mumby, Ltd., at £7,880, for the erection of the reinforced concrete columns and roof of the Moortown reservoir extension.

LINCOLN.—*Bridge.*—The City Council has accepted the tender of Messrs. W. Wright & Sons (Lincoln), Ltd., at £4,571, for widening the bridge at Newark Road in reinforced concrete.

MIDDLESEX.—*Bridge.*—The tender of Edwards Construction Co., at £12,025, has been accepted by the Middlesex C.C. for the erection of a reinforced concrete bridge over the Lee Navigation at the North Circular Road.

NEW MALDEN.—*Road Works.*—The Maldens and Coombe U.D.C. has accepted the tender of Messrs. W. Manders & Co., at £2,004 16*s.* 8*d.*, for making up with reinforced concrete the carriageway in Cotsfold Avenue; other tenders submitted were, Walker-Weston Co., Ltd., £2,352 7*s.* 8*d.*; Franks, Harris Bros., Ltd., £2,365 9*s.* 8*d.*; J. Mowlem & Co., Ltd., £2,387 11*s.* 5*d.*; J. Cronk & Sons, £2,897; R. H. Hartley & Co., £2,757 11*s.* For paving the footways in Cotsford Avenue with artificial stone the tender of the Patent Victoria Stone Co., at £622 1*s.* 8*d.*, has been accepted; other tenders for this work were, J. Mowlem & Co., Ltd., £653 3*s.* 9*d.*; W. Manders & Co., £671 17*s.*; Empire Stone Co., £709 3*s.* 6*d.*; J. Cronk & Sons, £746 10*s.*; R. H. Hartley & Co., Ltd., £823 16*s.* 6*d.*; Walker-Weston Co., Ltd., £995 6*s.* 8*d.*

NORTHAMPTON.—*Houses.*—The T.C. has accepted the tender of the Yorkshire Hennebique Contracting Co., Ltd., at £48,158, for the construction of 148 concrete houses.

RICHMOND (YORKS).—*Wall.*—The tender of Messrs. J. Busfield, Ltd., has been accepted by the West Riding of Yorkshire C.C. for the erection of a reinforced concrete retaining wall and other works near Thorpe House.

SALFORD.—*Foundations.*—The C.B. has accepted the tender of Messrs. E. B. Jones & Rawlinson, Ltd., at £1,689 13*s.* 10*d.* for the construction of concrete foundations at the Frederick Road power station.

SUNBURY-ON-THAMES.—*Bridge.*—The U.D.C. has accepted the tender of Messrs. K. Holst & Co., at £350, for the construction of a footbridge over the backwater at Sunbury.

TAUNTON.—*Concrete Houses.*—A tender of £399 per house has been accepted by the T.C. for the erection of 42 concrete houses.

New Companies Registered.

Rugby Portland Cement Co., Ltd. (206,971). Registered June 30. Cement and lime manufacturers. Nominal capital: £120,000. Permanent directors: N. Edyvean-Walker, 2, Albert-street, Rugby; H. A. Walker, 22, Surrey-street, Strand, W.C. 2; and three others.

Noel Rees, Ltd. (206,813). Registered June 23. 21 College Hill, E.C.3. Manufacturers of concrete slabs, blocks, tiles, etc. Nominal capital, £2,600. Directors: J. F. Burn, 24, Grosvenor-gardens, S.W.1; W. Noel Rees, 8, Northbrook-road, S.E.13.

Recent Patent Applications.

214,603.—G. F. Dampney: Concrete slabs.

214,962.—Ferguson Synstone Co.: Concrete blocks.

222,122.—E. Martin: Iron cement.

225,162.—C. W. Watson: Making concrete structures centrifugally.

234,144.—M. W .Harvey: Moulding concrete lintels and posts.

234,406.—N. Harrison: Concrete mixers.

234,545.—W. E. Clifton and Clifton-Ewart Construction Co., Ltd.: Moulding concrete structures.

234,586.—J. H. Harrison and N. Harrison: Moulds for casting concrete.

234,594.—E. L. Leeming: Reinforced concrete walls.

234,599.—C. J. Downing and W. B. Daniels: Building blocks and concrete buildings.

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234,707.—C. J. Tidy: Glazing of cement and concrete products.

235,006.—J. A. Eriksson: Manufacturing porous concrete.

235,080.—R. H. Kirk and H. Martiney: Manufacture of concrete slabs.

235,478.—V. Isern Y. Dominguez and F. P. De Los Cobos Y. Arda: Reinforced concrete sleepers.

235,606.—A. Ferguson: Processes for reducing ores and producing cement.

235,671.—C. Fitzsimon, jun.: Moulding concete blocks.

235,714.—A. J. Hodgkinson: Precast concrete blocks.

235,735.—A. A. Byrd: Monolithic concrete building construction.

Trade Notices.

In view of the great demand for rapid-hardening cement it is not surprising that other cement manufacturers have followed the lead of the Cement Marketing Co., Ltd., in the production of a cement which has such enormous possibilities in quick construction. Messrs. G. & T. Earle, Ltd., of Hull, have now put on the market an aluminous cement known as "Pelumina," a combination of "Pelican" (their well-known brand of Portland cement) and "Alumina." As with "Ferrocrete," it is possible to strike the shuttering in 48 hours on concrete work in which "Pelumina" cement is used. We have received a copy of a 30 pp. booklet, which Messrs. Earle have issued dealing with the new product, which contains a mass of information for those contemplating the use of a rapid-hardening cement, including analyses of Portland cement, "Pelumina" cement, and *ciment fondu*; notes on the construction of reinforced concrete buildings, roads and footpaths, piles, floors, cast stone, sea defences, etc.; and a comparison with the strength of "Pelumina" and the requirements of the British Standard Specification, and with Portland cement of twenty years ago. An appendix contains a number of testimonials and appreciations from Borough Engineers and contractors who have used the material.

Messrs. Brunner, Mond & Co., Ltd., inform us that the tests on sodium silicate as a concrete hardener, carried out at the National Physical Laboratory, and referred to in our August issue, were undertaken on their behalf, and that the figures showing the increased wearing properties of test pieces of concrete relate to specimens treated with their P.84 grade silicate. Messrs. Brunner, Mond & Co., have also sent us some literature they have had prepared on the subject which should be of value to all interested in hardening concrete and a specimen of concrete which has been partly treated with silicate and then tested on an abrading machine; this conclusively shows the merits of silicate, as while the untreated portion has worn down by three-eighths of an inch the treated portion shows no sign of wear.

We have received from Messrs. Crosby Lockwood & Son, of Stationers' Hall Court, London, E.C.4, a summary of press notices of their recent publications, including Mr. W. L. Scott's *Reinforced Concrete Bridges*. This summary is useful as showing the views of independent press reviewers, and not those of the publishers alone, and copies may be obtained free from Messrs. Crosby Lockwood.

The August issue of *The Edgar Allen News* (which will be sent free on request by Messrs. Edgar Allen & Co., Ltd., Imperial Steel Works, Sheffield) contains a paper by Mr. F. Bland on "A Century of Permanent Way," an account of the firm's exhibits at the New Zealand International Exhibition, and a list of tarnishing agents which have no effect on stainless steel.

Mr. Ormerod Mitchell has taken up the position of General Manager of the Ransome Machinery Company (1920), Ltd.

Change of Address.

Messrs. The Harvie Corporation, Ltd., have removed to new offices and showrooms at Broadway Buildings, Broadway, Westminster, S.W.1. (Telephone, Victoria 805; Telegraphic address, Harviproof, Sowest London.)

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EDITORIAL NOTES.

Fire Resistance of Concrete Columns.

IN this issue we give an abstract of the results of a comprehensive series of tests carried out in the United States to determine the fire-resisting qualities of various types and shapes of reinforced concrete columns. The investigations were undertaken by Messrs. W. A. Hull and S. H. Ingberg, and while their conclusions in some respects simply confirm facts which are already well-known the authors have gone very thoroughly into the question and set out the results in a manner which adds to their importance. Sixty-two test columns composed of different aggregates and with varying reinforcement were submitted to fire exposure of increasing intensity until failure or a period of four hours, and the strength of the columns at the end of this test compared with that of similar columns without the fire test. As will be seen from the table given, a very wide difference in the effects of the fire exposure was noted, and, as was to have been expected, this was chiefly due to the difference in the aggregates of which the various test-columns were made. It is shown that the tendency to spall in fire test and in

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fires in buildings is much greater when the concrete is made of siliceous aggregates, owing to the expansion properties of certain minerals which predominate in this type of aggregate, and that the behaviour of quartz and granite when heated to the inversion point of quartz (1,064 deg. Fahr.) renders concrete containing these classes of minerals specially susceptible to damage by fire. Tests on exactly similar columns showed that whereas those made with a limestone or other calcareous aggregate successfully withstood fire exposure for six hours, those containing 60 per cent. or more of siliceous aggregate failed after 2½ hours.

These tests also show that round columns are not necessarily superior to square columns in point of fire resistance. In columns made from non-spalling aggregates, such as limestone, blast-furnace slag, and trap rock, the results do not indicate any advantage of one form over the other. The least favourable results were those from fire tests of round columns with both longitudinal and spiral reinforcement and made from spalling aggregates, including gravels, very high in quartz content and containing large proportions of quartz and granite. The round hooped column proved to be particularly susceptible to fire damage except when made with concrete in which the aggregate was of the non-spalling type. On the other hand, round columns without hooping but with spalling aggregates gave somewhat better results than square columns of the same type. If columns are made from concrete with a non-spalling aggregate there is thus no advantage, in point of fire resistance, in the round over the square column, and in the case of spalling aggregates the fire resistance problem cannot be solved by the use of round instead of square columns ; this applies particularly to hooped columns.

The tests with plain concrete columns indicate that from the standpoint of safety during the fire they are as safe under their working load as the reinforced columns are under the higher working load allowed for the different types, although where made with aggregates of the spalling type light metal reinforcement and ties in the outer protection are highly desirable.

In the matter of size of column the results indicate decrease in resistance with decrease in size, although with the same thickness of protection the higher ratio of concrete considered as protection to that considered as load bearing helps to reduce the actual unit stress imposed in the case of the smaller columns, except for hooped columns made with siliceous aggregates, where the disruption and spalling of the protective concrete left the load supported on the unprotected column core.

It is demonstrated that concrete columns require the protection of heat-insulating material of some kind, just as steel columns do, although failure of unprotected concrete columns on exposure to fire is not as imminent because of the lower conductivity of the material. In cases where spalling aggregates are used, plain concrete covering the steel is not an efficient protective material. Either the protective concrete should be provided with light metallic reinforcement to prevent its dislodgment or some other material should be substituted for it. On the other hand, plain concrete covering the steel has been shown to be efficient and satisfactory as protective material if made from an aggregate in the non-spalling class.

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Foreign Competition in Building.

A GOOD deal of attention has recently been given to the fact that a German firm has secured a contract for a housing scheme for the Dublin municipal authorities. The houses are to be built in concrete, and the total of the contract amounts to over £100,000. The press reports of this item of news have, however, given rise to an erroneous impression of the facts. We were told, for instance, that concrete blocks were to be imported into Ireland from Germany, and that the German tender was far below that of any local firm, and those interested in building works began to wonder whether we had entered an era when building would no longer be a "sheltered" industry. But the time has not yet arrived, and we doubt if it ever will, when concrete blocks can be made on the Continent and shipped here to sell at competitive prices with the home-made product. The system to be used by Messrs. Kossel, of Bremen, who have secured the Dublin contract, is a monolithic one, and the only material to be imported from abroad is the special type of shuttering they use. As to the price on which this contract was secured, it is interesting to note that it was only 2 or 3 per cent. below the nearest British tender, and, further, that in the case of other housing contracts in Ireland, of equal magnitude, for which this firm competed, they have in each case been beaten on price. But the fact that a Continental firm can tender for housing work in the British Isles at a figure approximating to British prices deserves serious attention, and a careful study of the methods employed with a view to ascertaining how this is possible might be made with advantage by our own firms concerned with the erection of concrete houses.

Safety of Building Operatives.

As we expected when they were first issued by the Home Office in 1921 the Draft Regulations for Building Operations under the Factory and Workshop Act met with considerable opposition from both employers and employed, and it has taken four years to reconcile these differences. A revised draft has, however, now been agreed by the chief associations of employers and trade unions concerned and it is presumed that they will be put in force in the near future. The contentious clauses have been omitted or altered, and as they stand now the duties they impose on builders so far as protection of moving machinery, the provision of suitable and efficient scaffolding, etc., are concerned amount to nothing more than the precautions which would naturally be taken by any employer with a regard for the life and limbs of people working for him. Such precautions are now practically always taken, and while in the great majority of cases the new regulations will impose no hardship or involve any alteration of practice, they will do good by compelling the careless employer to take steps to prevent accidents occurring.

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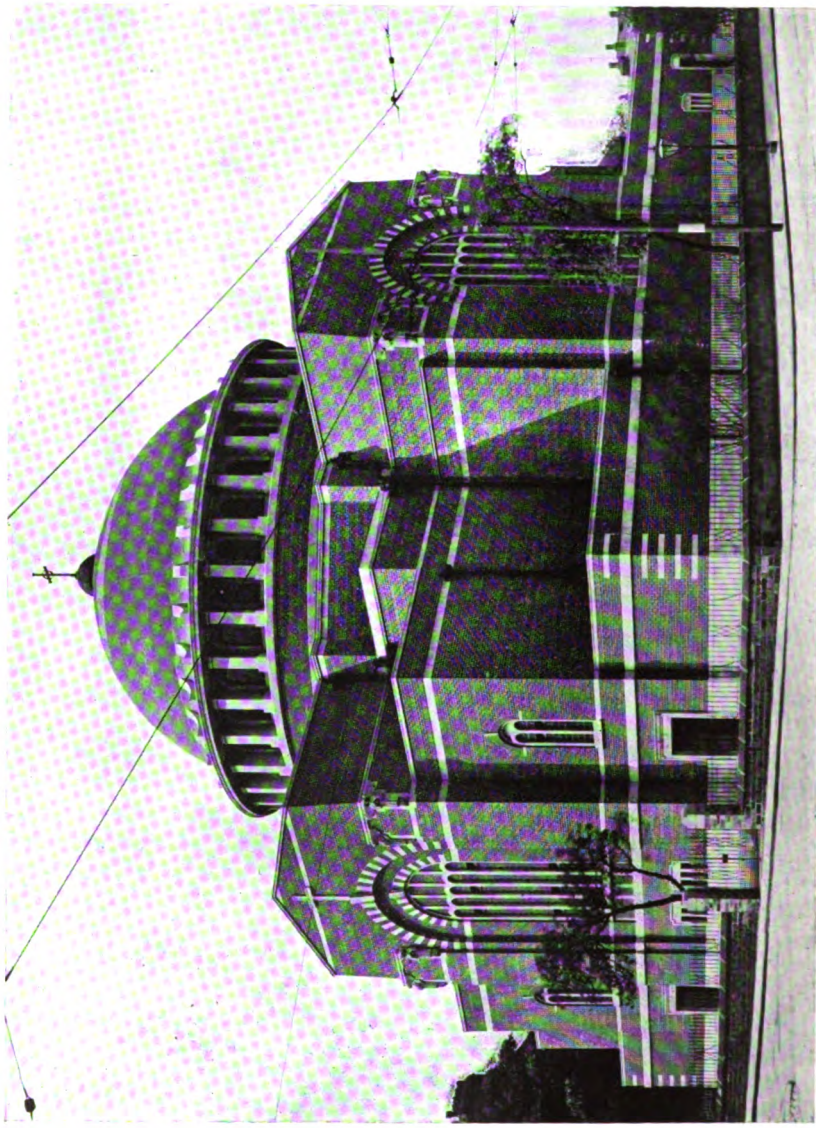

Mr. E. Bower Norris, A.R.I.B.A., Architect; Mr. Burnard Geen, M.Inst.C.E., Consulting Engineer.

Fig. 1.—Church of St. John, Rochdale. (*See p.* 541.)

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Church of St. John, Rochdale.

THE church illustrated and described in this article was executed to the general design of Mr. E. Bower Norris, A.R.I.B.A., of Messrs. Hill, Sandy & Norris, of Manchester, and is an interesting example of the application of reinforced concrete to a structure of a complicated character.

As will be seen from *Fig.* 1 the domed church is of Byzantine detail, massive but simple in character, the perspective sketch shown in *Fig.* 3 illustrating the campanile intended as part of the final building but omitted for the present on account of cost.

On the outside the work below the springing of the dome is brick faced enriched with artificial stone dressings, but above that level the concrete is exposed.

The general arrangement of the church is shown in *Fig.* 4, from which it is seen that the main loads are carried upon eight main vertical frames, of varying detail, which are seen in course of construction in *Fig.* 2. These eight frames are supported on reinforced concrete footings, which in turn rest upon mass concrete footings below a standard level, and carried down to varying depths to provide a foundation in dense sand below the site, the pressure being limited to 1½ tons per sq. ft., with which load no settlement has been recorded.

Between each pair of frames at a height of 30 ft. there is a barrel vault 31 ft. in diameter finished on the inner edge with a heavy arch beam to receive the pendentives carrying the main dome. The barrel vaults, 6-in. thick, are tied in on the outside by ties passing through the windows and obscured by the pattern of the window framing and forming part of it.

The main dome, 54 ft. internal diameter, is 4½-in. thick at the crown, increasing to 5½-in. thick at the springing and extending to pendentives 9-in. thick, the diameter of the pendentives being 67 ft. 6 in.

At the base of the dome 36 windows are provided, which together with the main side windows below effectively light the church.

The treatment of the base of the dome required careful consideration, and to provide the column and arch treatment with the minimum weight the following method was adopted: The moulded columns were made hollow 2-in. thick, pre-cast at the contractors' works, with connecting bars projecting in both directions, and erected in position and connected with the *in situ* work at the base of the dome when that was cast. By this means an excellent finish was obtained, and a considerable saving in dead-weight effected.

The entering ribs on the exterior of the dome were pre-cast in similar fashion, and above the arched window openings the *in situ* concrete was cored out to reduce the dead-load as much as possible.

The internal scaffolding to the dome was arranged so that the hoist rose out of the centre, leaving the centre for a measuring space from which the dome could be accurately set out on the ground, the setting-out being then transferred to the dome springing level.

During the progress of the work very severe weather was experienced, and great care was required to check the work continuously and ensure that the pre-arranged accuracy was being maintained.

The various floors, flat roofs, gallery, etc., call for no special comment and presented no special difficulty. The general effect of the completed interior is shown in *Fig.* 5.

The details of the whole work were of an exceptionally complicated character needing great care both in preparation and execution.

The Portland cement used in the work was B.S.S. slow setting cement, which upon test gave average results as under:—

Neat.		Sand.	
7 days	28 days	7 days	28 days
lbs. sq. in.	lbs. sq. in.	lbs. sq. in.	lbs. sq. in.
843	948	437	486

The whole of the concrete was machine mixed, and exceptional care was taken to obtain the best proportions in order to produce concrete which would justify the high working stresses adopted in the actual work.

Three classes of concrete were used, as under:—

6 to 1 ballast concrete, in mass concrete footings, below the reinforced concrete footings.

27 cub. ft. granite to 13½ cub. ft. sand

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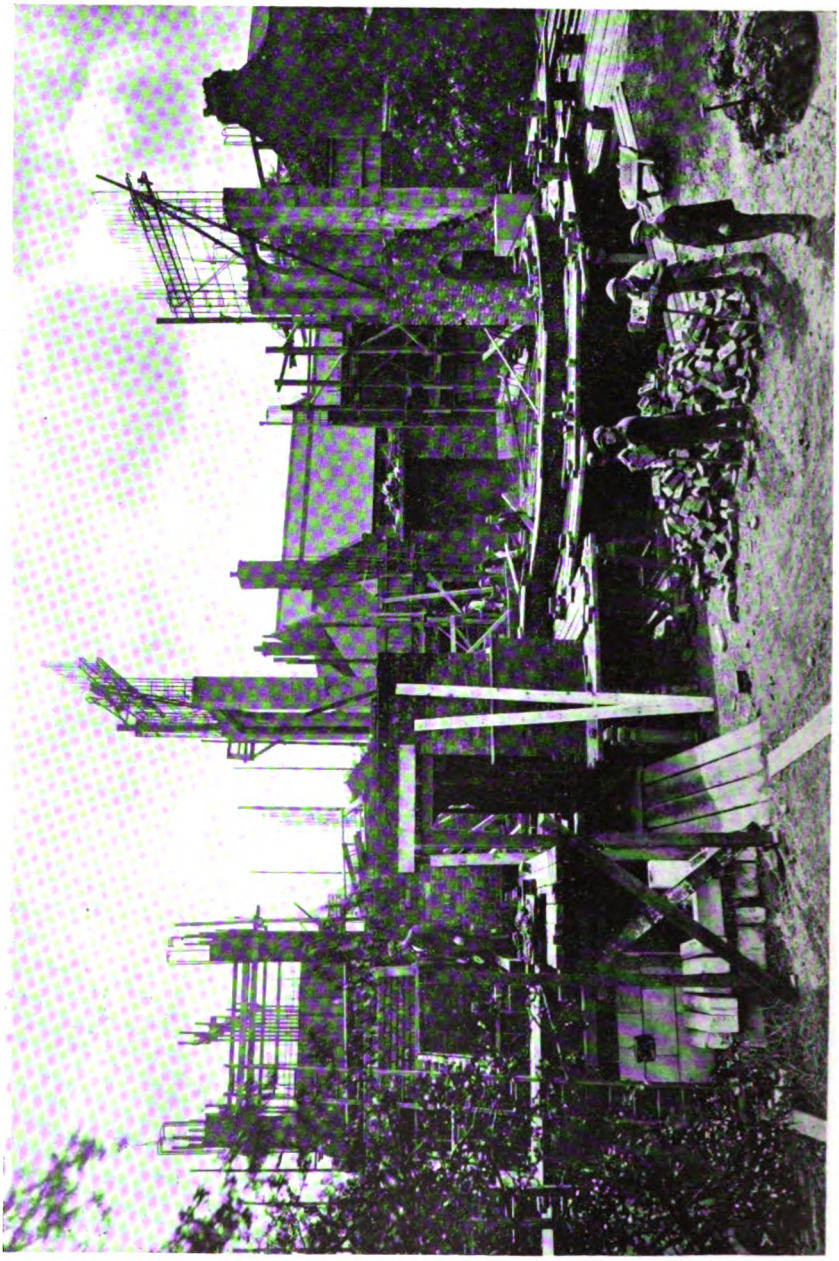

Fig. 2.—Church of St. John, Rochdale : Main Vertical Frames in Course of Erection.

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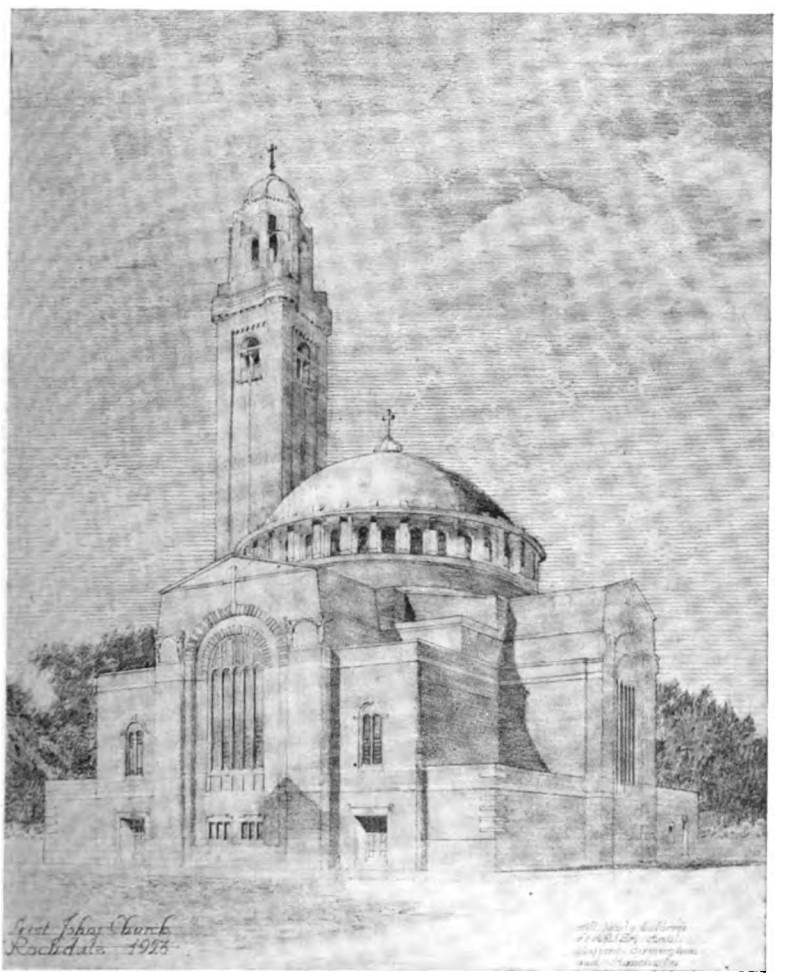
to 6½ cwts. cement, in all reinforced concrete up to the barrel vaults.

27 cub. ft. granite to 13½ cub. ft. sand to 7 cwts. cement in barrel vaults, dome, and pendentives. The granite was Welsh granite, clean, and well graded on the site, varying in size from ¾ in. to ¼ in.; the voids were repeatedly tested and the proportions of sand and cement slightly varied from time to time as necessary. The sand employed was Leighton Buzzard sand, clean, sharp, and well graded in size. Special care was taken with the question of the quantity of water used for mixing, and it was found in general by tests on cubes that the strength of the resulting concrete was much reduced with an increase of water beyond the required amount.

The following results of crushing tests on 6-in. cubes, selected from a considerable number of cubes made from the concrete used in the work and stored under similar conditions to the actual



Perspective Sketch.

Fig. 3.—Church of St. John, Rochdale.

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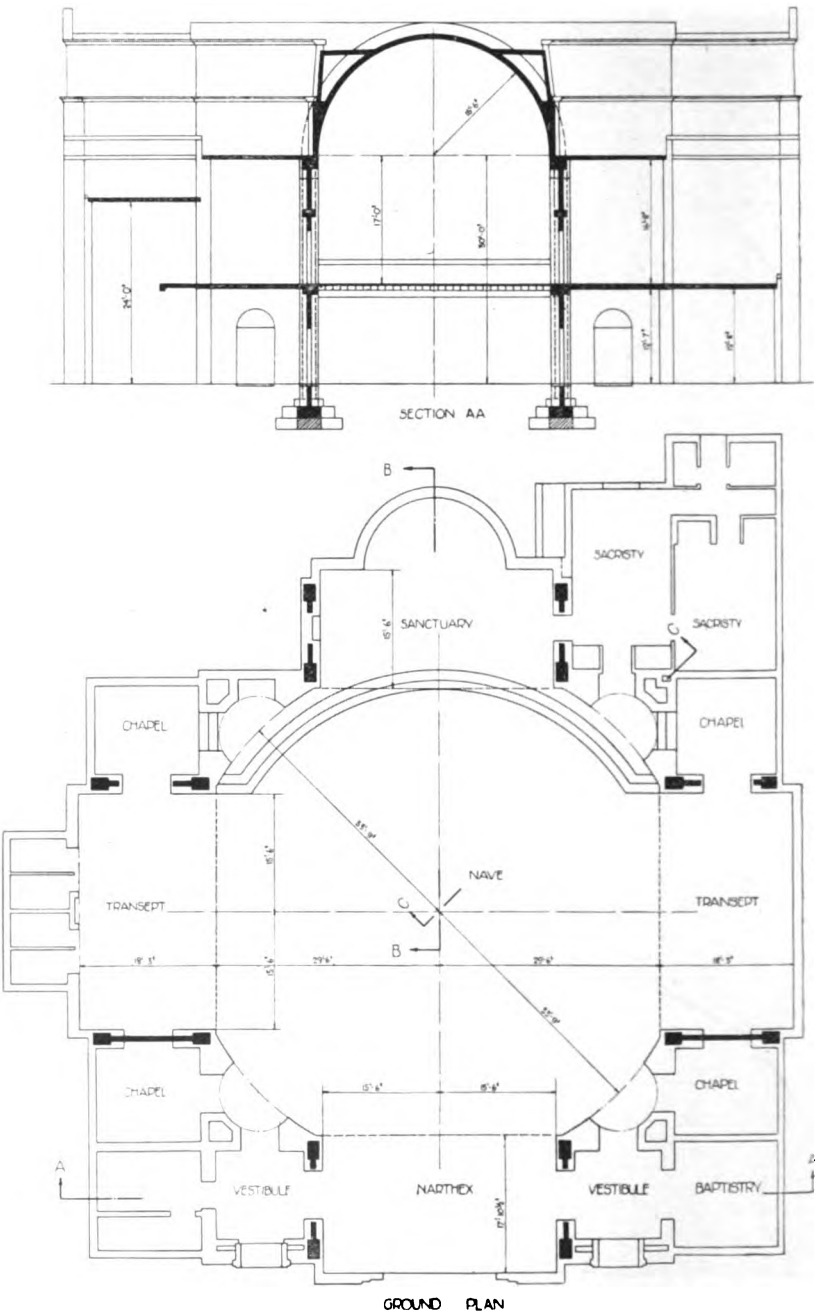

Fig. 4a.—Church of St. John, Rochdale.

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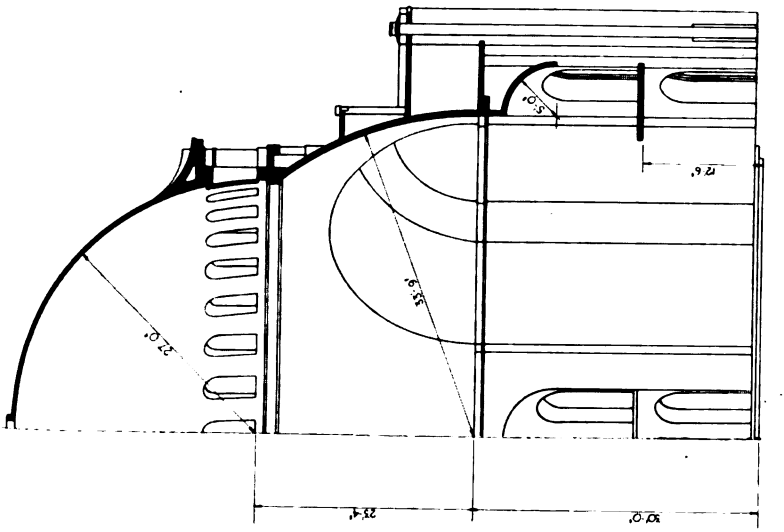

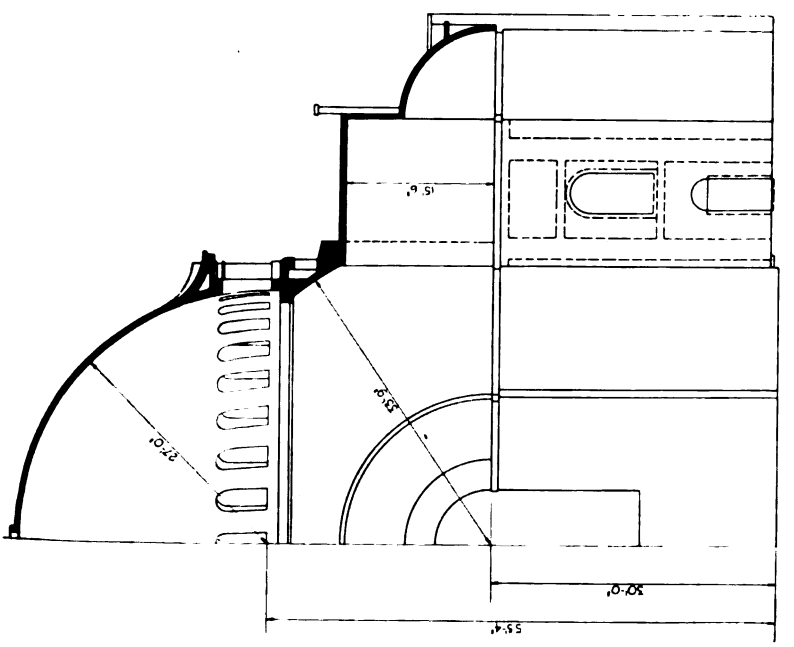

Fig. 4b.—Church of St. John, Rochdale.

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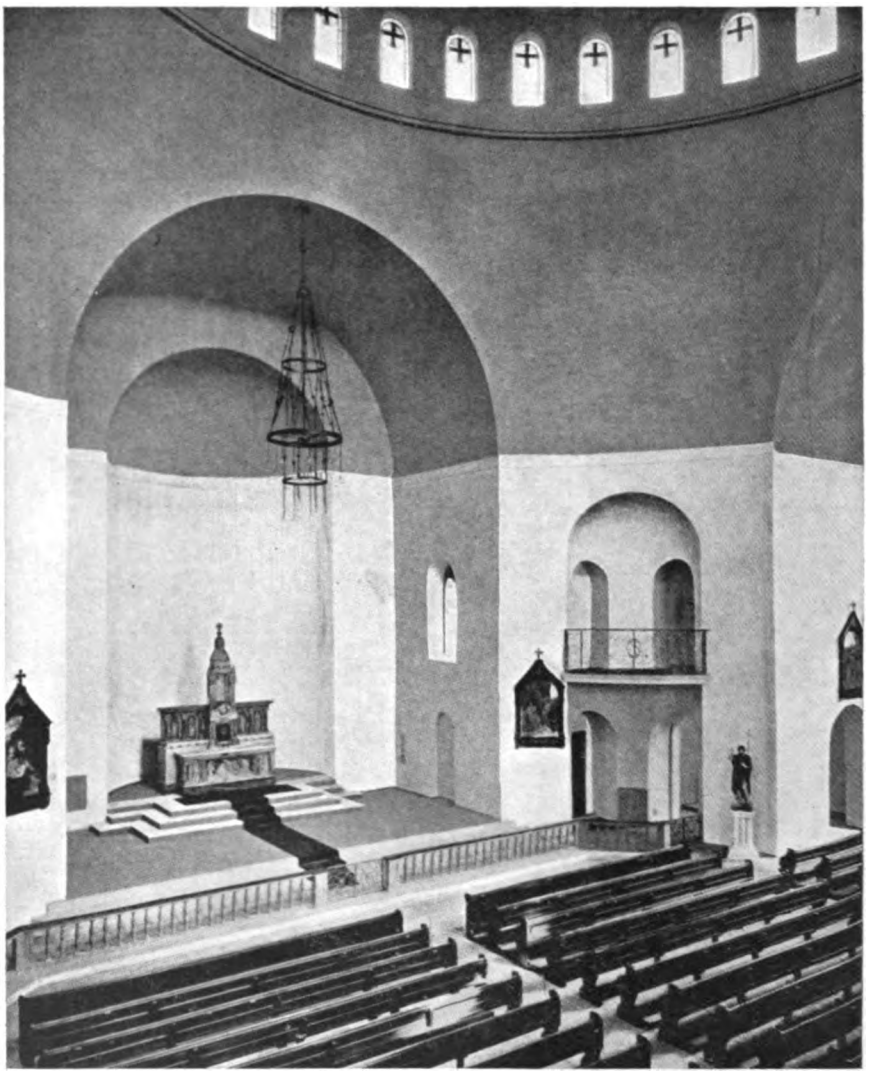

Mr. E. Bower Norris, A.R.I.B.A., Architect;
Mr. Burnard Geen, M.Inst.C.E., Consulting Engineer.

Fig. 5.—Church of St. John, Rochdale: Interior View.

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concrete in the work, will give an idea of the exceptionally high standard of strength obtained.

It will be noted that in several cases the actual force employed to crush the 6 in. cubes was in excess of 100 tons, or 400 tons per sq. ft.; and the final result for the cube marked (*) was lost for the reason that it was made upon a testing machine limited to 100 tons compression. When it was seen that these high results were being obtained the remaining cubes were crushed in another testing machine of greater capacity. The variation in stress between the various ages is of interest, and the last cube on the list is remarkable. It was unfortunate that the one cube marked (*) could not be crushed, as it would have indicated whether the cube giving the figure of 7,526 lbs. per sq. in. which was made at the same time and from the same mix was a fair average.

Assuming that this had been proved to be the case, it raises the point whether when exceptionally high results such as these are being obtained one should expect only a very much smaller appreciation with age than usually applies to cubes having an average strength in compression of from 2,000 to 3,000 lbs. per sq. in.

The dome has proved to be entirely watertight since its completion, and the work has entirely justified the architect's decision to omit any external covering of asphalte, copper, or the like, and an examination of the structure after 18 months of life showed it to be perfect in every way.

Age in Days.	Weight. Lbs. per cubic foot.	Crushing Strength Calculated to Tons per Sq. Ft.	Lbs. per Sq. In.	Remarks.			
33	153	400	6,200	27 c. ft.:	13½ c. ft.:	6 cwts.	
33	152	383	5,951	do.	do.	do.	
90	153	498	7,738	do.	do.	do.	
33	144	299	4,657	do.	do.	7 cwts.	
33	145	330	5,132	do.	do.	do.	
90	142	335	5,213	do.	do.	do.	
46	151	346	5,388	do.	do.	do.	
45	149	383	5,950	do.	do.	do.	
45	151	484	7,526	do.	do.	do.	made on same day from same mix
45	151	400	6,222*	do.	do.	do.	
548	148	542	8,428	do.	do.	do.	

The specialists for the design and construction of the reinforced concrete work were Empire Stone Co., Ltd., for whom Mr. Burnard Geen, M.Inst.C.E., of Westminster, acted as consulting engineer. The reconstructed stonework is "Empire" stone.

The "Concrete Year Book."

MAY we ask those of our readers who have not yet returned the forms for their entry in the Directory Section of the 1926 Edition of "The Concrete Year Book" to do so at once, in order that they may be included? If any readers in the British Isles whose names should appear in this Directory, either as consulting engineers specialising in reinforced concrete or as firms catering for the industry, have not received a form a copy will be sent on application to the Editor, 20, Dartmouth Street, Westminster, S.W.1.

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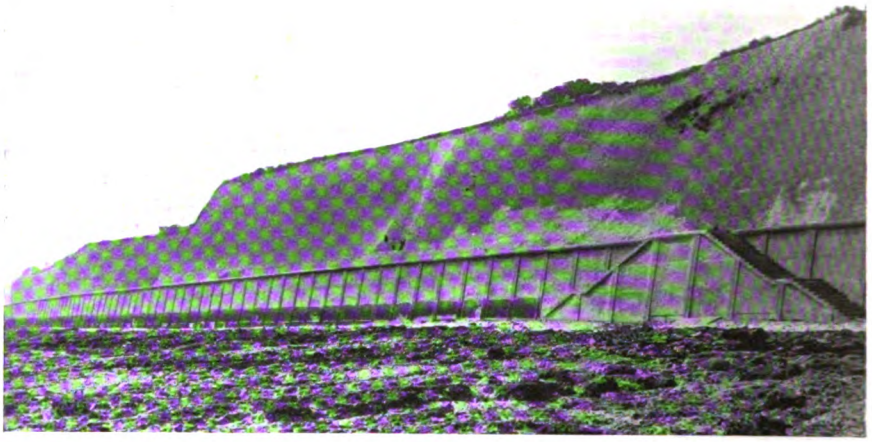

Fig. 1.—General View.

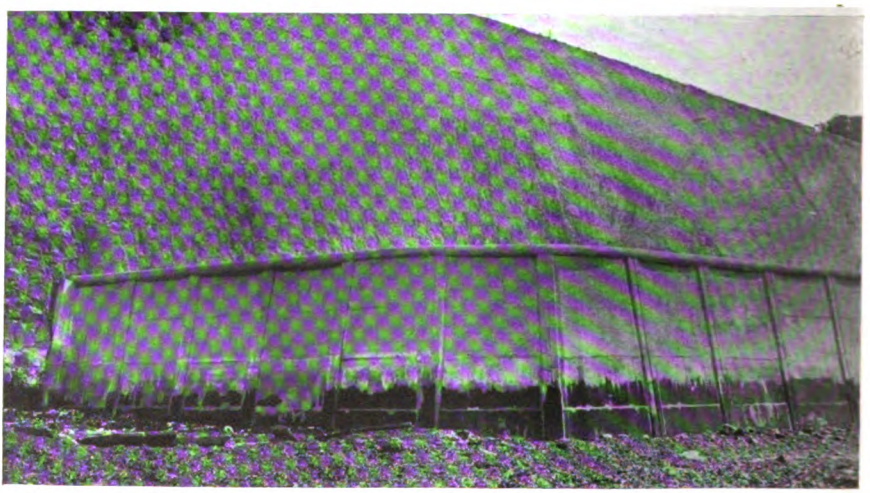

Fig. 2.—A Corner View.

Reinforced Concrete Sea Wall at Ramsgate. (*See p.* 549.)

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Reinforced Concrete Sea Wall at Ramsgate.

THIS sea wall, just completed, is built at the foot of the cliffs at Ramsgate opposite the St. Lawrence Estate. It is 3,000 ft. long, 17 ft. high above foreshore, and retains a promenade 50 ft. to 120 ft. wide formed by chalk filling obtained from the cliffs.

The wall is composed of pre-cast counterforts, slabs, and anchor beams. The counterforts are 10 ft. apart and of a triangular shape, the front part having grooves on both sides in which slide the slabs forming the front of the wall. The bottom side supports the anchor beams, which support the filling at the back of the wall and prevent this from over-turning.

The main advantages of this type of construction are :—

(1) Cheapness in concreting.

(2) The conditions under which the steel is placed are such as to ensure accuracy.

(3) The concrete is well matured before being put in contact with seawater.

(4) The quality of the concrete when the parts are pre-cast can be tested before being placed in position.

(5) The wall can be built between tides, without the use of a cofferdam.

6. Speed of execution is increased.

Figs. 3 and 4 show the making of the different parts. The work was started in August, 1924, and *Fig.* 5 shows the erection of the last counterfort in April, 1925. A total of 3,000 ft. of wall was completed, the work being carried out during one of the worst winters experienced in Ramsgate for the past thirty years.

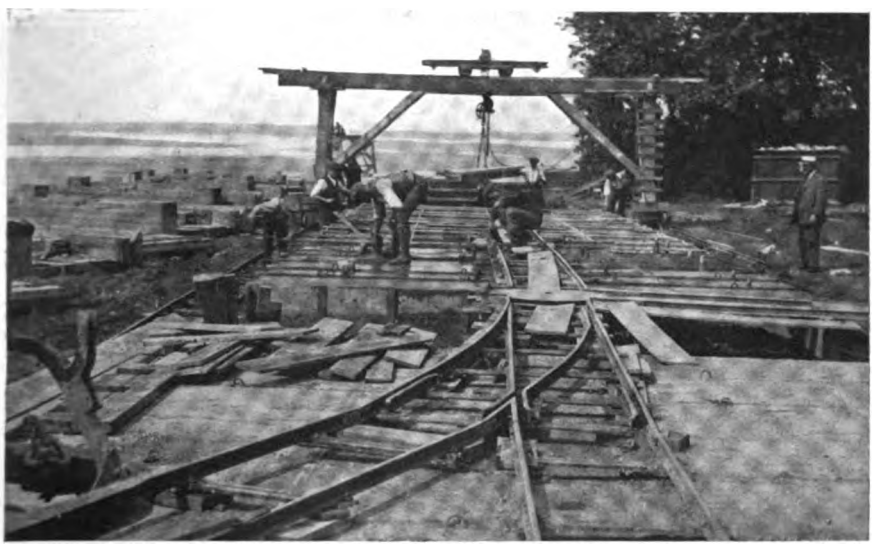

Fig. 3.—Making Pre-cast Units.

All the parts were concreted on top of the cliff, which is from 85 to 90 ft. high. There is 18 cubic ft. of concrete and about 1 cwt. of steel per foot run of wall.

Messrs. The Indented Bar and Concrete Engineering Co., Ltd., were responsible for the design, and Messrs. The Monnoyer British Construction Co., Ltd., were the contractors for the Ramsgate Corporation.

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Fig. 4.—Making the Pre-cast Units.

Fig. 5.—Erection of Last Counterfort.

Reinforced Concrete Sea Wall at Ramsgate. (*See p.* 549.)

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Fire Resistance of Concrete Columns.

A BULLETIN containing the results of exhaustive investigations on the fire-resistance of concrete columns, carried out by Messrs. W. A. Hull and S. H. Ingberg, has been published by the Department of Commerce of the United States Bureau of Standards.

Fire tests were made of 62 columns under working load, and 16 comparable columns were tested to failure in compression without fire test. The thickness of concrete or other material considered as protection was 1½ or 2½ in. Concrete aggregates of a wide range in mineral composition were introduced.

The columns were submitted under normal working load to fire exposure increasing in intensity according to a predetermined time-temperature relation until failure or until the end of four hours, when additional load was applied up to the limit of the furnace equipment (600,000 lbs.).

A wide difference in fire effects on different columns was noted and was determined as due in large part to difference in mineral composition of the concrete aggregates used. Quartz, chert, or granite induced spalling and cracking of the concrete when subjected to fire, while concrete made with calcareous aggregates, such as limestone or calcareous gravel, showed little visible effects even from very severe fires. As made with trap rock or blast furnace slag, the fire effects were intermediate between those above indicated. Shape of section or type of column reinforcement were shown to be of minor importance. It was further shown that with concrete protection of siliceous aggregates or of plaster subject to cracking and spalling a large improvement could be effected by placing metal ties, such as wire mesh, in the protection. Columns made with siliceous aggregates and thus protected withstood, under working load, fires of four hours duration, and at their conclusion in the heated condition loads from two to over four times the working load.

Aggregates.—The results of this investigation confirm the conclusion that the tendency to spall in fire tests and in fires in buildings of concrete made from certain types of aggregate is due to the expansion properties of certain mineral constituents which predominate in those aggregates. The expansion behaviour of quartz and of granites when heated to temperatures as high as the inversion point of quartz (1,064 deg. F.) is such as to render concrete containing large proportions of either or both of these classes of minerals especially susceptible to damage by fire.

Shape, Design, and Size.—It has been shown by this work that round columns are not necessarily superior to square columns in point of fire resistance. In columns made from non-spalling aggregates, such as limestone, blast-furnace slag, and trap rock, the results do not indicate any advantage of one form over the other. The least favourable results were those from fire tests of round columns with both longitudinal and spiral reinforcement and made from spalling aggregates, including gravels, very high in quartz content and containing large proportions of quartz and granite. The round hooped column proved to be particularly susceptible to fire damage, except when made with concrete in which the aggregate was of the non-spalling type. On the other hand, round columns without hooping but with spalling aggregates gave somewhat better results than square columns of the same type. If columns are made from concrete with a non-spalling aggregate there is no advantage, in point of fire resistance, in the round over the square column. In the case of spalling aggregates, the fire resistance problem cannot be solved by the use of round instead of square columns, and this applies particularly to hooped columns.

The tests with plain concrete columns indicate that from the standpoint of safety during the fire they are as safe under their working load as the reinforced columns are under the higher working load allowed for the different types, although where made with aggregates of the spalling type light metal reinforcement and ties in the outer protection are highly desirable.

In the matter of size of column the results indicate decrease in resistance with decrease in size, although with the same thickness of protection the higher ratio of concrete considered as protection to that considered as load bearing helps to reduce the actual unit stress imposed

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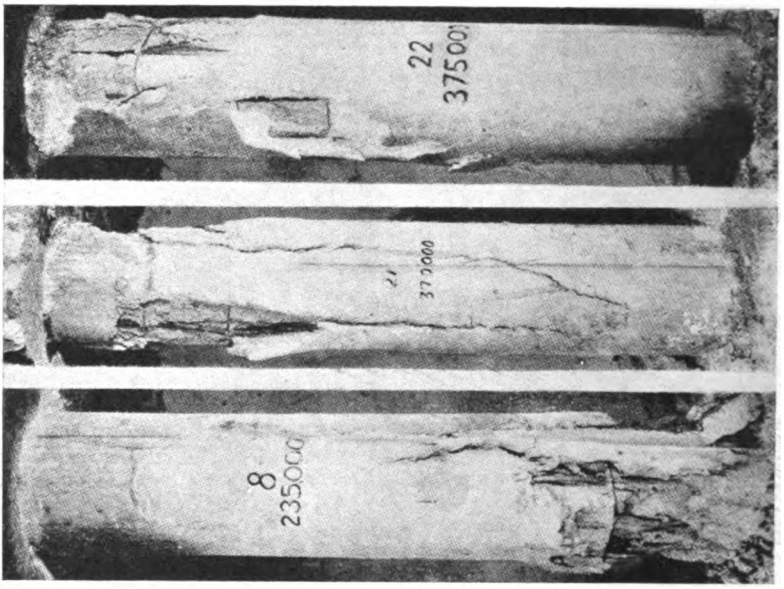
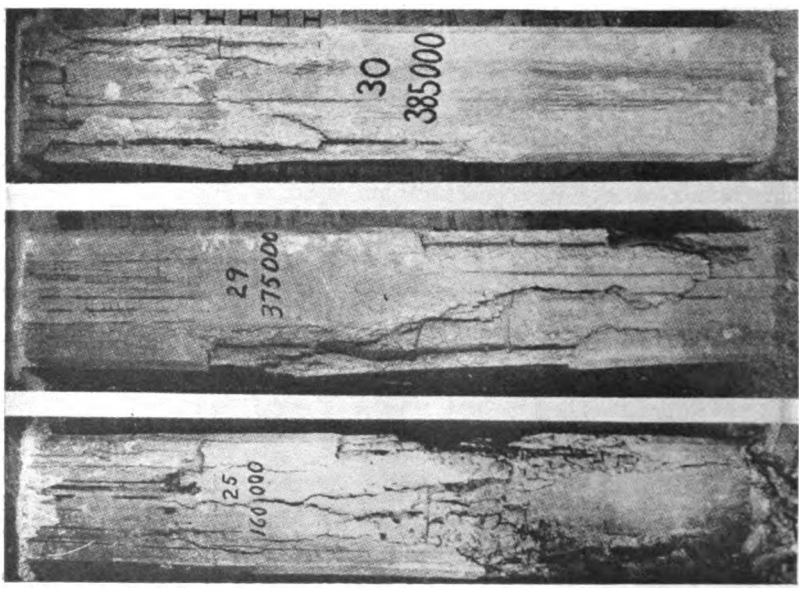


Round Vertically Reinforced Columns after Failure when Load was Increased at End of Four-Hour Fire Test.



Square Vertically Reinforced Columns after Failure when Load was Increased at End of Four-Hour Fire Test.

Fire Tests of Concrete Columns.

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in the case of the smaller columns, except for hooped columns made with siliceous aggregates, where the disruption and spalling of the protective concrete left the load supported on the unprotected column core.

Protection.—It has been demonstrated that concrete columns require the protection of heat-insulating material of some kind, just as steel columns do, although failure of unprotected concrete columns on exposure to fire is not as imminent because of the lower conductivity of the material. In cases where spalling aggregates are used, plain concrete covering the steel is not an efficient protective material. Either the protective concrete should be provided with light metallic reinforcement to prevent its dislodgment or some other material should be substituted for it. On the other hand, plain concrete covering the steel has been shown to be efficient and satisfactory as protective material if made from an aggregate in the non-spalling class.

The evidence of the fire-test results indicates that in columns made from spalling aggregates the expansion stresses in the load-bearing portion of the column contribute to the loss in strength more than in columns made from non-spalling aggregates if temperatures as high as 575 deg. C. or over are attained in the load-bearing concrete. In other words, the expansion behaviour, which has a tendency to produce spalling, may weaken the column even though no spalling actually occurs. However, it seems necessary if appreciable fire damage is to be prevented to insulate columns and other structural members well enough to prevent the steel and load-bearing concrete from reaching these temperatures whether the aggregate used be of a spalling or non-spalling type. Judging from the results of this investigation, concrete made from a spalling aggregate may be as nearly immune to fire damage as that made from a non-spalling aggregate provided it is so protected that the temperatures in the steel and load-bearing concrete will be kept reasonably low—say, below 932 deg. F.—which appears feasible where necessary or desired. The problem of providing the necessary insulation is, of course, simpler when a non-spalling aggregate is used.

The most important function of columns in buildings subjected to fire is to support load, and thereby prevent local or general collapse. Reinforced concrete buildings as a type present desirable general fire-resistive characteristics in being incombustible and not subject to immediate collapse on exposure to fire. It is shown in these tests that while there is a wide range in the fire resistance of concrete columns, depending mainly on the mineral constituents of the aggregates employed or the method of protecting the load-bearing portion, even the less favourable results obtained indicate an inherent fire resistance that must be recognised when the main concern is the ability to function satisfactorily as load-bearing members in fires of the intensity and duration to which buildings of a given type and housing given occupancies may be subjected.

Portions of the investigation were devoted to the development of means for reducing such fire effects to a minimum and show that this can be done and is possibly a profitable thing to do under almost all conditions. The ability to come through fires without failure under working load does not imply freedom from damage to the load-bearing portion as well as to the protecting material. Such damage under exceptional conditions may be such as to render complete replacement or major repair necessary.

Fire-resistance Periods.—The fire-resistance period is a convenient unit to which results of fire tests can be reduced and made available for use. It is derived from the results of tests, with suitable margins taken for variations in material, workmanship, design and size of columns obtaining in building construction to which the results may be applied but not obtaining in the relatively few test specimens and construction conditions that could be introduced. With safe units thus established and an estimate made of the duration of fires possible in a building of given type and occupancy, column types and protections can be chosen adequate for the conditions to be met.

Method of Derivation.—The fire-resistance periods are derived with main reference to ability to perform satisfactorily during the fire exposure. For columns that failed during the four-hour fire test or took only a small increment

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Fire-Resistance Periods of Concrete Columns.

[Concrete mixture to be not leaner than 1 : 2 : 4 Portland cement, sand, and coarse aggregate, by volume. Details of design and loading to conform with generally accepted standards.]

Type of column.	Shape.	Minimum area of solid material.	Coarse aggregate.	Protection.		Fire-resistance period.
				Description.	Thickness.	
		Sq. in.			Inches	Hours.
Plain concrete	Round	200	60 per cent. or more quartz, chert, or granite.	Plain concrete, same as column . .	1½	2½
Vertically reinforced.	Square	200	do.	do.	1½	2½
Do. . .	Round	200	do.	do.	1½	3
Do. . .	Round or square.	200	Trap rock or blast-furnace slag.	do.	1½	3½
Do. . .	do. .	200	Limestone or calcareous gravel; quartz, chert, or granite not more than 10 per cent.	do.	1½	4
Laterally and vertically reinforced.	do. .	110	60 per cent. or more quartz, chert, or granite.	do.	1½	1½
Do. . .	do. .	200	do.	do.	1½	2½
Do. . .	do. .	200	Trap rock or blast-furnace slag.	do.	1½	4
Do. . .	do. .	200	Limestone or calcareous gravel; quartz, chert, or granite not more than 10 per cent.	do.	1½	6
Do. . .	do. .	200	60 per cent. or more quartz, chert, or granite.	Concrete as column reinforced with expanded metal or wire mesh.	1½	3½
Do. . .	do. .	250	do.	Plain concrete same as column . . .	2½	3
Do. . .	do. .	250	do.	Concrete as columns reinforced with expanded metal or wire mesh.	2½	6
Do. . .	do. .	250	do.	1½ in. plain concrete as column covered with 1-in. Portland cement or gypsum plaster reinforced with expanded metal or wire mesh; plaster proportions not leaner than 1 : 2½ by volume with not over one-half part lime.	2½	6
Do. . .	do. .	250	Limestone or calcareous gravel; quartz, chert, or granite not over 10 per cent.	do.	2½	6
Do. . .	do. .	250	60 per cent. or more quartz, chert, or granite.	½ in. concrete as column covered with a 2-in. thick unreinforced gypsum mould within which column is cast.	2½	3
Do. . .	do. .	250	do.	Same as above, except gypsum mould reinforced with expanded metal or wire mesh.	2½	6
Do. . .	do. .	200	do.	½ in. concrete as column covered with 1-in. unreinforced Portland cement or gypsum plaster not leaner than 1 : 2½ by volume with not over one-half part lime; surface hacked or column cast in metal-lath form.	1½	3
Do. . .	do. .	200	do.	½ in. concrete as column covered with 1-in. unreinforced Portland cement-cinder plaster, 1 : 3 by volume; surface hacked or column cast in metal lath form.	1½	4
Do. . .	do. .	200	do.	½-in. concrete as column covered with 1-in. unreinforced Portland cement asbestos and sand plaster of proportion 1 : 1½ : 2 by volume; surface hacked or column cast in metal lath form.	1½	5
Do. . .	do. .	200	do.	½ in. concrete as column covered with 1 in. of gypsum, hydrated lime, kieselguhr, plaster, of proportion 1 : ⅔ : 2 by volume, reinforced with 2-in. wire mesh or equivalent.	1½	6

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of load at its conclusion the safe fire-resistance period is taken as two-thirds of the average time to failure of a group of similar columns under the standard fire conditions of the test, but not higher than four-fifths of the lowest resistance developed by any column within the group. Periods are taken at intervals of one-half hour up to four hours, an even value of the resistance period being taken if the computed value is not more than one-fourth of the interval below or three-fourths of the interval above the given period value.

For columns that withstood under working load the four-hour fire test the four-hour classification is given if the load sustained while hot at the end of the fire test is not less than 50 per cent. of the strength of comparable columns not subjected to fire tests. For columns failing at the end of the four-hour fire test under lower loads corresponding reductions in the fire-resistance periods assigned them are made. For columns withstanding a greater load while hot at the end of the four-hour fire test the five-hour classification is given if the strength while hot is not less than 55 per cent. of that of comparable columns not subjected to fire and not less than 70 per cent. of such strength when tested cold after subjection to the maximum load of the furnace equipment (3,480 lbs./in.^{2}) while hot. The six-hour classification is given if the strength while hot is not less than 55 per cent. of that of similar columns not fire tested, and when tested cold after fire test not less than 80 per cent. of the unfire cold strength. Periods over six hours are not significant since the six-hour standard fire exposure is considered more severe than the maximum fire exposure occurring in buildings under any but the most exceptional conditions.

The method of deriving the resistance periods has been chosen as most acceptable for the test conditions of this investigation and should not necessarily be taken as generally applicable to investigations where different test conditions are present.

Table of Fire-resistance Periods.—The periods derived from the results of this investigation according to the method outlined above are given in the Table. The resulting periods cover a range from one and one-half to six hours or over, depending mainly on the column size and type, concrete aggregates used, and thickness and type of protection. It is seen that for columns not smaller than 16 in. in outside diameter or 15 in. square a variation in safe-resistance period from two and one-half hours to six hours obtains, mainly from difference in mineral composition of the concrete aggregates employed. Siliceous aggregates with which the lower periods were obtained are taken to have not less than 60 per cent. of quartz, chert, or granite, and the calcareous aggregates with which the highest resistance obtains not more than 10 per cent. of the same minerals. No definite information has up to the present been obtained on the fire resistance of concrete made with gravels and other aggregates containing siliceous material in amount intermediate between these limits, and no periods for concrete columns made with such aggregates are here given, although presumably they would be intermediate between those given for highly siliceous aggregates and those for calcareous aggregates. The trap rock and blast-furnace slag concrete are seen to occupy a position intermediate between concrete made with aggregates predominantly siliceous and calcareous.

The following periods indicate the effectiveness of adding secondary reinforcement in the concrete protection where siliceous aggregates are used, the resistance periods increasing from two and one-half hours without the reinforcement to three and one-half hours with the mesh or expanded metal in the 1½-in. protection and from three hours to six hours or more where reinforcement is added in the 2½-in. protection. It is seen that little is gained by increasing the thickness of protection without adding the secondary reinforcement. Six-hour resistance is obtained by adding 1 in. of reinforced plaster outside of the 1½ in. of concrete or by casting the column in reinforced gypsum forms 2 in. thick.

All the latter methods involve protections of 2½ in. total thickness, which may be objectionable under many conditions. The concluding part of the table gives periods for protections where the plaster replaces an equal thickness of protective concrete, thereby reducing the total thickness of the protection to 1½ in. It is seen that only a minor gain

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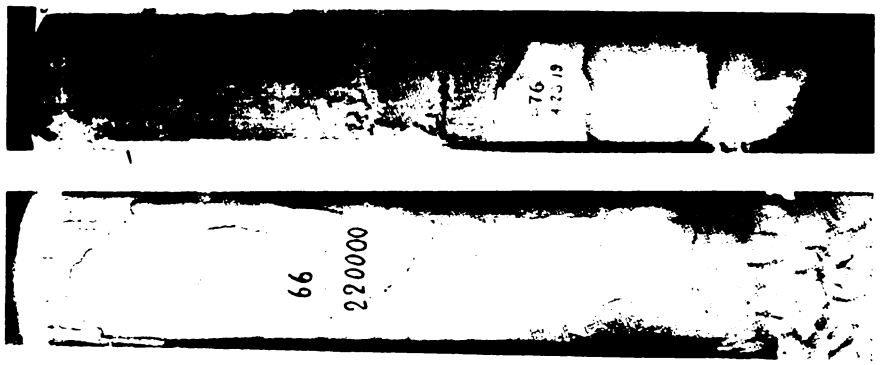
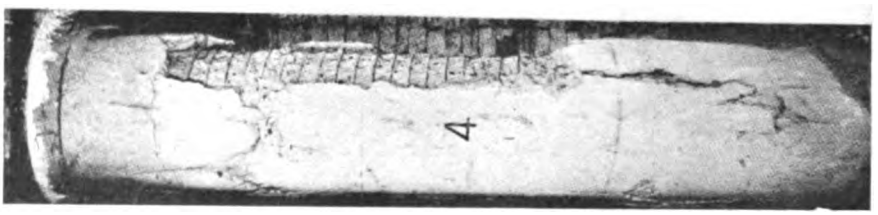
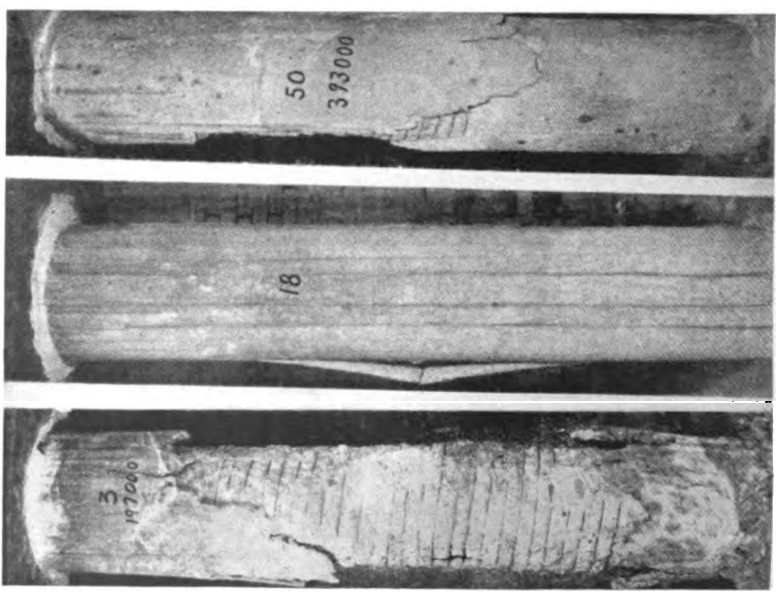




Columns protected by Plaster after Failure when Load was increased at End of Four-Hour Test.



Spirally and Vertically Reinforced Columns after Failure when Load was increased at End of Four-Hour Test.

Fire Tests of Concrete Columns.

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is obtained by replacing 1 in. of siliceous gravel concrete with unreinforced Portland cement or gypsum plaster. Somewhat greater resistance is obtained with unreinforced cement-cinder plaster and cement-asbestos-sand plaster, due to somewhat better insulating properties and greater ability to remain in place. The highest results were obtained with a reinforced gypsum-lime-kieselguhr plaster held in place by the wire mesh. Considerable gain in resistance would obtain with the other plasters if similarly reinforced.

Comparison with other Investigations.—In a jointly conducted investigation of the fire resistance of building columns, six reinforced concrete columns were included in the fire-test series and three were subjected to fire and water tests. The columns were round or square vertically reinforced and round spirally hooped and vertically reinforced, the concrete aggregates employed being trap rock and dolomitic limestone, except in the columns for the fire and water tests, portions of which were made with siliceous gravel concrete. All columns had a 2-in. protection of concrete the same as in the column, without reinforcement.

Two of the trap-rock concrete columns withstood the standard fire test under working load for periods between seven and eight hours, and the third sustained a load at the end of the eight-hour fire test about 25 per cent. in excess of the working load. The limestone concrete columns all withstood the eight-hour fire test under working load and, at its conclusion, loads averaging two and one-fourth times the working loads. With both aggregates, little cracking and almost no spalling occurred during the fire test. In the one-hour fire test preceding the water application in the fire and water test the trap rock and limestone concrete were little affected, while the siliceous gravel concrete cracked and spalled, exposing portions of the reinforcement in the case of the hooped column. The five-minute water application from a 1⅛-in. nozzle under 50 lbs. pressure carried away the concrete damaged during the preceding fire exposure and some of the adjacent portions, the vertical bars of the square columns being generally exposed and the bars and spiral hooping in the portion made with siliceous aggregates in the case of the round columns. After cooling the columns sustained at failure loads averaging four times the design working load. The condition of the columns after the fire and water treatment was not such that they would be subject to early failure in case of a recurring fire, with the limitation on permissible percentages of reinforcement prescribed by current standards of practice. Recurring fires are likely to be considerably less severe than an unchecked fire, considering the effects of the preceding fire and the water application. The conclusion was therefore reached that the fire and water tests developed nothing that would modify conclusions from the results of the fire-endurance tests. This is significant, since in the series reported in this paper no fire and water tests were made, and the results from the other investigation indicate that for reinforced concrete columns of ordinary design conclusions can be safely based on results from fire tests alone.

Comparing the resistance periods deduced from the results of the other investigation, vertically and combined laterally and vertically reinforced columns made with trap-rock aggregate with 2 in. of concrete protection were given a period of five hours as against three and one-half and four hours for those in the present investigation with 1½-in. concrete protection. With limestone concrete columns the comparison stands eight hours against six hours on the same basis. The conclusions from the two investigations are believed consistent, considering the thinner protection of columns in the present investigation, and the intention to have the fire-resistance periods apply to columns near the minimum size used in buildings and of the range in reinforcement percentages allowed by current design standards.

No experimental investigations of the fire resistance of concrete columns other than the one above referred to have been made under conditions comparable with the present investigation, the few tests made being under no load or loads considerably below working load values and with fire exposures of different intensity.

Intensity and Duration of Fires.—To apply the fire-resistance periods derived from this investigation per the preceding section and the Table to the

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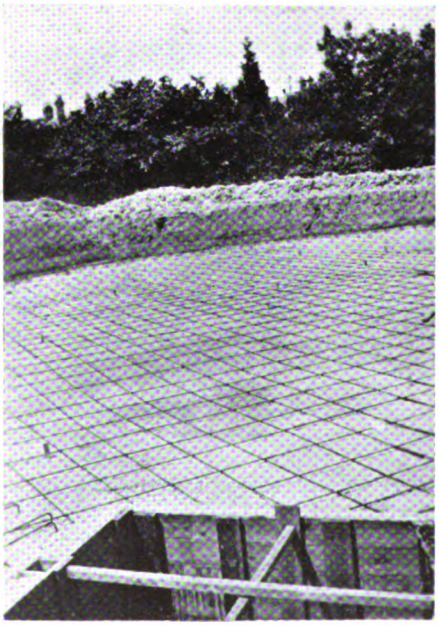
different building types and occupancies some knowledge is necessary of the intensity and duration of fires that destroy the building contents and combustible trim. Reinforced concrete columns will almost invariably be used in combination with concrete or other incombustible structural floors and floor framing. There may, however, be a considerable amount of combustible material in the top or finish floor, partitions, and interior trim. The combustible contents may vary in amount from a minimum with residential, institutional, and office occupancies to a range from minimum to near maximum with manufacturing and warehouse premises.

The greatest amount of material and consequently the severest fires are to be expected in warehouses and storage buildings where fires apparently equivalent in intensity and duration to the four or five-hour fire exposure in the tests have occurred. For residential, institutional, and office occupancy some experimental investigations recently conducted at the Bureau of Standards, as also observation of the effects of fires in buildings, indicate that fires in such buildings do not generally exceed a severity equivalent to the first hour of the fire-test exposure, and severity greater than the one and one-half hour exposure would obtain only in portions used for storage or similar purposes. Warehouses and factories would accordingly present a range of severity equivalent to the one or two-hour fire-test exposure at the lower limit and approximately to the four-hour period where material in the larger amounts are present.

This estimate of probable intensity and duration of fires, while admittedly tentative, is believed to have sufficient value to warrant its presentation as an aid in applying the results of this investigation to the common building types and occupancies and appears to be confirmed by fire experience to the extent that definite information has been obtainable from this source.



View of Foundation, Showing Valve Pit in Foreground.

Reinforcement Around Edge of Tank.

New Gas-Holder Foundation at Haywards Heath. (*See p.* 559.)

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New Gas-Holder Foundation at Haywards Heath.

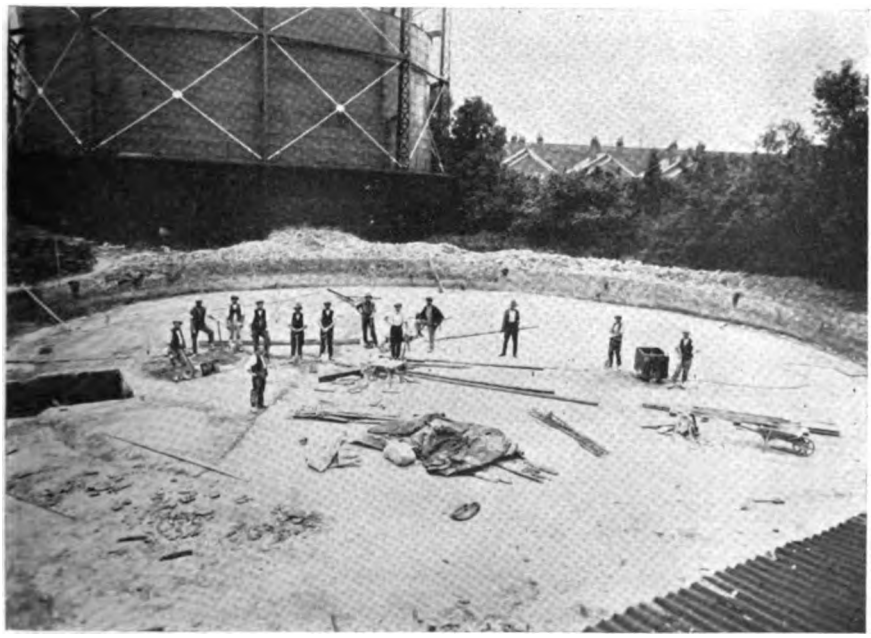
THE foundation for the new 250,000 cubic feet spiral-guided gas-holder now in course of erection at the Haywards Heath Gas Works is of reinforced concrete, prepared to the plans and specification of the Engineer, Mr. A. E. Whitcher.

A large portion of the ground had been previously made up and some traces of bog and blue marl were found, but on reaching the depth of 4 ft. the clay was quite solid. A 2-in. layer of mass concrete was then laid over the bottom of the pit, which was 87 ft. 6 in. in diameter. The concrete was made of 1 part Portland cement, 3 parts clean sharp sand, and 6 parts clean gravel to pass $\frac{3}{4}$-in. mesh. The object of this layer was to fill any inequalities in the site as excavated and to form a clean bed on which to lay out the reinforcement. It also ensures that the main reinforced concrete slab, which is depended on for strength, shall be of a uniform thickness and that none of the cement grout in the layer shall sink away into the ground underneath.

The reinforced slab is 5 in. in thickness, and the proportions of the concrete used were 1 part Portland cement, 2 parts clean sharp sand, and 4 parts $\frac{3}{4}$-in. gravel. Plain round mild steel reinforcing rods of $\frac{1}{2}$ in. diameter were used. These were laid so as to form 15-in. squares over the bottom of the pit, as shown in one of our illustrations. Running round the edge of the slab was a trench 3 ft. wide and 9 in. deep, in which the reinforcing rods were bent and formed into a strong mattress. This gave a total depth of 14 in. of reinforced concrete, which bears the weight of the sides of the tank.

The steel tank is 85 ft. 6 in. in diameter and 25 ft. in depth. It is five tiers in depth, the bottom tier being $\frac{1}{2}$ in. thick, the second $\frac{7}{16}$ in., the third $\frac{3}{8}$ in., the fourth $\frac{5}{16}$ in. and the fifth $\frac{1}{4}$ in. The bottom plates are $\frac{1}{4}$ in. in thickness.

Messrs. The F. C. Construction Co., of Derby, have satisfactorily carried out the whole of the work.



New Gas-Holder Foundation at Haywards Heath.

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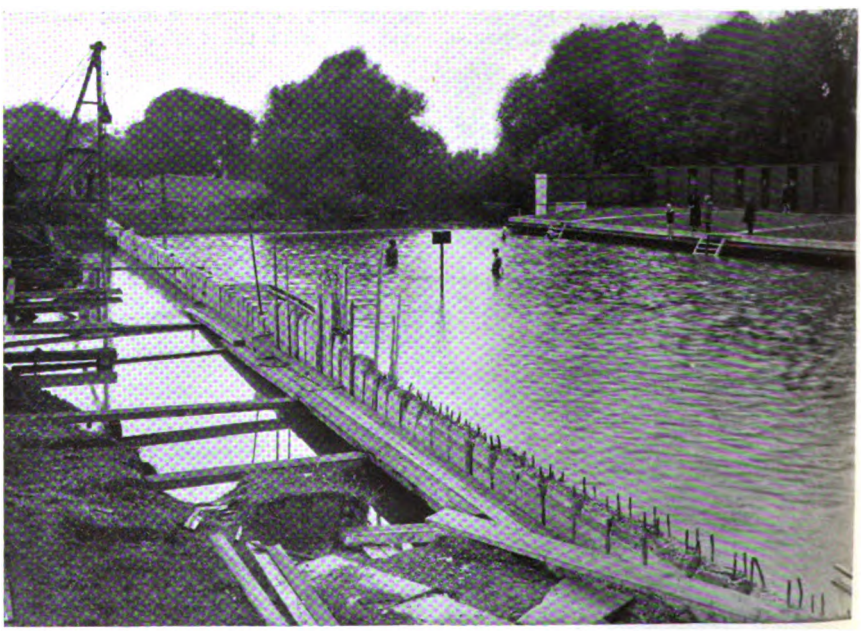
Completed.



[*Mr. E. A. Strickland, Borough Engineer.*

During Construction.

New Open-air Bath at Windsor. (*See p.* 561.)

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Open-Air Bath at New Windsor.

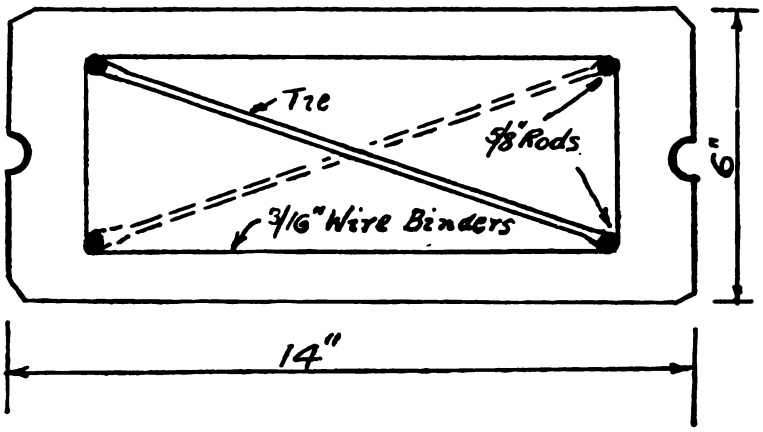
A LOOP on the river Thames at Windsor has been used for some considerable time as an open-air swimming bath. The banks of the stream were retained in the first instance by timber camp-sheeting behind piles, and as the timber rotted it was replaced by mass concrete walls, foundations for which were formed of concrete in sacks. With the periodical flooding of the land by the Thames overflowing the bank cavities were formed behind the wall, and consequently the foundations were considerably undermined, with the result that sections of the wall collapsed into the river and a large amount of erosion took place.

The Windsor Borough Council decided to form a reinforced concrete wall, and their Engineer, Mr. E. A. Strickland, prepared several schemes; on costing them it was found that reinforced concrete sheet piling would give the best results. The Council approved of the reinforced concrete sheet piling, and the work was accordingly put in hand.

The length of the stream which was to be piled for the baths was 235 ft. The depth of the water varied from 3 ft. to 8 ft. The subsoil being of chalk on a sloping bed, the piles were made of various lengths so as to penetrate into the hard strata underneath. The section of the pile was 14 in. by 6 in., each pile being reinforced with No. 4 $\frac{5}{8}$-in. diameter steel rods tied at 6-in. centres for the major portion of the pile, with $\frac{3}{16}$-in. diameter steel wire close to the head and shoe; the ties were fixed at 3-in. centres. At 2 ft. 6 in. centres diagonal ties were fixed in the pile cages to make them rigid and prevent buckling during the stiff driving. Each pile was shod with a cast-steel shoe which had a projecting nib which ran down the groove formed in the side of the pile and helped to keep the piles in alignment. These grooves were afterwards filled with cement grout held in with jointing.

The mixture of concrete used in constructing the pile was 1 part of "Ferrocrete" rapid-hardening cement, 2 parts sand, and 4 parts $\frac{3}{4}$-in. gravel. The piles were driven shortly after they had been moulded, after which the concrete was stripped from the steel for a depth of 12 in. from the head. On top of the piles a continuous coping beam was constructed, and from the steel in this beam the piles were anchored back at suitable intervals to blocks of concrete in the ground well behind the angle of repose of the soil. Underneath the coping beam provision was made for a running board and chains.

The completed work gives a bath 235 ft. long and 60 ft. wide, with a depth of water from 3 ft. to 8 ft. The contract was carried out by Messrs. Concrete Piling, Ltd., of 68/9, Windsor House, Victoria Street, S.W.1, under the supervision of the Borough Engineer, and the Council are satisfied that the use of reinforced concrete will eliminate a considerable amount of expense in that there will be no expenditure for repairs or renewals.



SECTION OF PILE.

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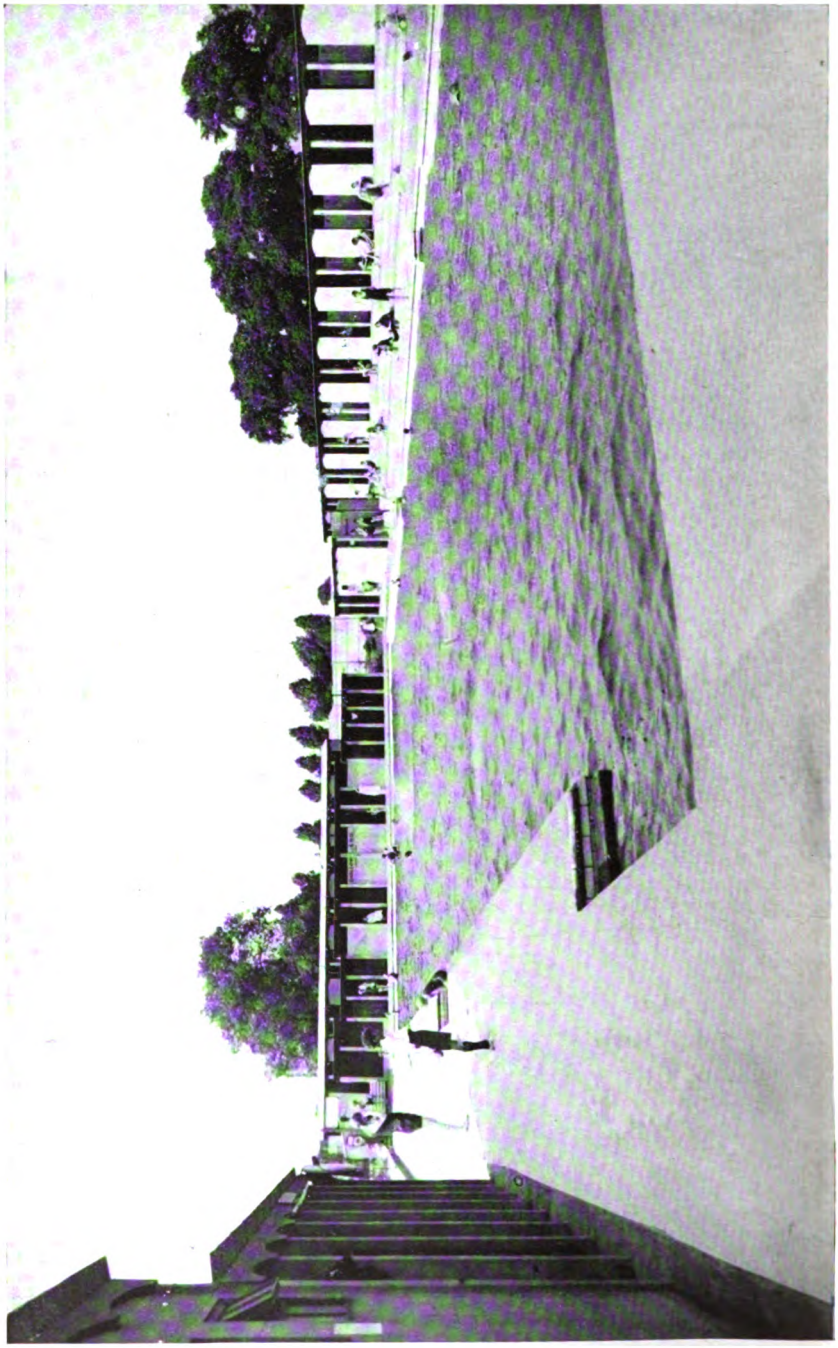

Open-air Swimming Bath at Lewisham. (*See p.* 563.)

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Open-Air Swimming Baths.

PEN-AIR swimming baths in reinrced concrete are increasing in popurity and in number, and soon few dustrial towns of importance will be ithout one or more of such baths. ne bath recently built at Lewisham has en constructed by the Borough Council ith unemployed labour. It has a water rface of 150 ft. by 60 ft. and is from e design of the Borough Engineer, r. W. F. Owsley. The walls vary in ight from 3 ft. to 7 ft. 6 in., the total ater capacity being 275,000 gallons. ne site of the bath is the river Ravensurne at Bellingham, which was diverted r the purpose, and particular attention id to be given to the floor, which was nstructed after the form of a raft. eel wire lattice reinforcement, supplied Messrs. Johnson's Reinforced Conete Engineering Co., Ltd., was used, id the floor, which has a thickness of in., has maintained watertightness nce the opening of the bath three years go.

The aggregate for the walls and floors as 6 to 1 Thames ballast and cement, e finishing coat being ½ in. of 2 to 1 id ¼ in. of 1 and 1 cement and sand. feature of the bath is that the steps e inset in the decking, and do not oject into the pool. The total cost the work was about £7,000.

Designed and carried out by Mr. Harley Heckford, M.Inst.C.E., the Borough Engineer and Surveyor, an open-air swimming bath has been constructed in Millwall Recreation Ground by the Poplar Borough Council. The scheme was adopted as relief work, and it was therefore decided that the structural part of the scheme should be executed in reinforced concrete. The greater portion of the remainder of the work consisted in excavating to form the bath, the material removed being embanked and consolidated to form the bath surround. This method dispensed with the necessity for carting away material from the site. A retaining wall 580 ft. in length, around the site, serves as a base for a 6-in. concrete boundary wall of an average height of 8 ft. The enclosure is 200 ft. in length, and 90 ft. wide, comprising an area of 2,000 yds., exactly double that of the actual bath, which is 150 ft. long and 60 ft. wide, with a varying depth of from 4 ft. 6 in. to 8 ft., the actual depths of water, when full, being 3 ft. 8 in. to 7 ft. 2 in.

For our illustrations we are indebted to a well-produced brochure on Concrete Swimming Baths issued by the British Portland Cement Association, and which will, we understand, be sent free on request to those interested.

Open-Air Swimming Bath at Aldenham.

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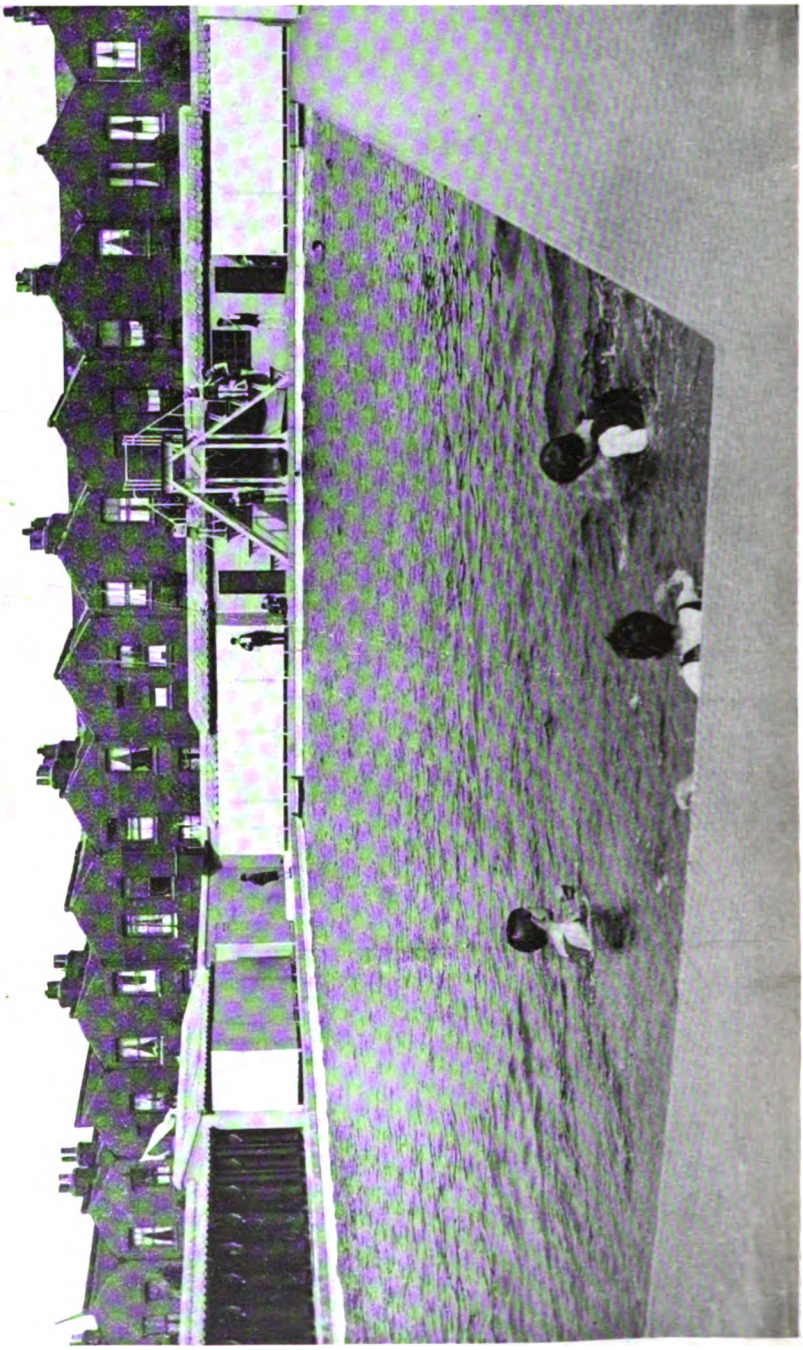

Open-air Swimming Bath at Poplar. (*See p.* 563.)

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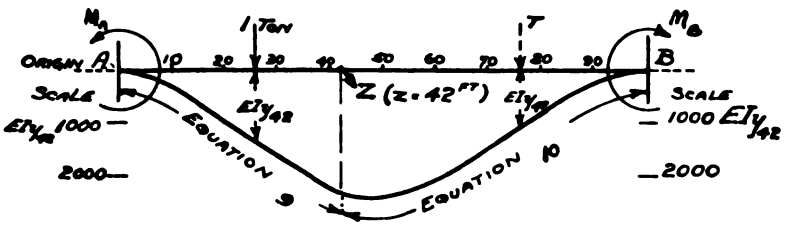
The Deflection of and the Unit Influence Lines of Deflection for Girder Beams.

(*a*) Simple, (*b*) Encastre and (*c*) Continuous: and for which the Product *EI* is Constant.

By H. W. Coultas, M.Sc.(B'ham.); B.Sc.(Leeds).

(*Concluded.*)

The Unit Influence Line of Deflection for a Section *Z* distant *z* from the origin as a unit load crosses the fixed beam *AB* (Fig. 5).



UNIT INFLUENCE LINE OF DEFLECTION × EI
FOR SECTION Z 42 FT. FROM THE ORIGIN

Fig. 5.

Let the unit load be between A and Z, i.e. $z > nl$. The equation for the deflection at Z will be obtained by adjusting equation (8) by making $x = z$ and where n will be the variable.

$$\therefore EIy_z = -(1 - 3n^2 + 2n^3)\frac{z^3}{6} + \frac{nl}{2}(1 - n)^2 z^2 + \frac{(z - nl)^3}{6} \quad . \quad . \quad (9)$$

Let the unit load be between Z and B, i.e. $nl > z$. The equation for the deflection at Z will be obtained by altering equation (7) by making $x = z$ and n the variable. Then

$$EIy_z = -(1 - 3n^2 + 2n^3)\frac{z^3}{6} + \frac{nl}{2}(1 - n)^2 z^2 \quad . \quad . \quad . \quad . \quad (10)$$

The influence line of deflection can thus be drawn by using equations (9) and (10).

For a number of concentrated loads crossing the span, the deflection at Z will be $y_z = \frac{1}{EI}\Sigma Wy$, as explained previously for a simple beam.

Again, the area of the unit deflection diagram above a length of uniformly distributed loading times the rate of loading will give the deflection at Z as a uniformly distributed load moves over the span. Depending upon the position of, and the length of the distributed load, the deflection at Z can be obtained by integrating either or both equations (9) and (10) with respect to nl between the proper limits.

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Maximum deflection due to a uniformly distributed load of length longer than the span. It will obviously occur when the whole span is covered: and it will be the sum of the deflections when AZ is only covered and when ZB is only covered. Deflection at Z when AZ covered:

$$EIy^a_{zu} = EI\int wy_z \,.\, d(nl) = EIw \,.\, l\int y_z \,.\, dn$$

$$= wl\left[\int_{n=0}^{n=\frac{z}{l}} -\frac{z^3}{6}(1 - 3n^2 + 2n^3)\,.\,dn + \frac{z^2 l}{6}\int_{n=0}^{n=\frac{z}{l}} n(1-n)^2\,.\,dn + \int_{n=0}^{n=\frac{z}{l}} \frac{(z-nl)^3}{6}\,.\,dn\right]. \quad (11)$$

Deflection at Z when BZ only covered

$$EIy^b_{zu} = wl\left[-\frac{z^3}{6}\int_{n=\frac{z}{l}}^{n=1}(1 - 3n^2 + 2n^3)\,.\,dn + \frac{z^2 l}{2}\int_{n=\frac{z}{l}}^{n=1} n(1-n)^2\,.\,dn\right] \quad . \quad (12)$$

The sum of y^a_{zu} and y^b_{zu} will give the deflection at Z.

SPECIAL CASE.

The deflection at the centre of the beam when the whole span is covered. $AZ = ZB = \frac{l}{2}$.

The maximum deflection will be twice that considering only AZ covered when $n = \frac{1}{2}$.

$\therefore$ $EIy^a_{zu} + EIy^b_{zu} = 2$ times equation (11) working between the limits of $n = \frac{1}{2}$ and $n = 0$.

$$= 2\left\{-\frac{wl^4}{48}\left[n - n^3 + \frac{n^4}{2}\right]_0^{\frac{1}{2}} + \frac{wl^4}{8}\left[\frac{n^2}{2} - \frac{2n^3}{3} + \frac{n^4}{4}\right]_0^{\frac{1}{2}} - \frac{wl^4}{384}\left[(1-2n)^4\right]_0^{\frac{1}{2}}\right\}$$

$$= 2\left\{-\frac{wl^4}{48}\left[\frac{1}{2} - \frac{1}{8} + \frac{1}{32}\right] + \frac{wl^4}{8}\left[\frac{1}{8} - \frac{1}{12} + \frac{1}{64}\right] + \frac{wl^4}{384}\right\} = \frac{2wl^4}{768} = \frac{wl^4}{384}. \quad . \quad (13)$$

The deflection at the centre with the beam wholly covered

$$= \frac{wl^4}{384EI}$$

Deflections of Continuous Beams carrying a single unit load in only one bay.

The supports being all of the same height, EI = constant.

Fundamental Conditions:

(*a*) The ends of the beam are hinged;

(*b*) The beam is held down at each support point.

Case 1.

A girder continuous over two spans AB, BC of length l_1 and l_2 respectively (*Fig.* 6).

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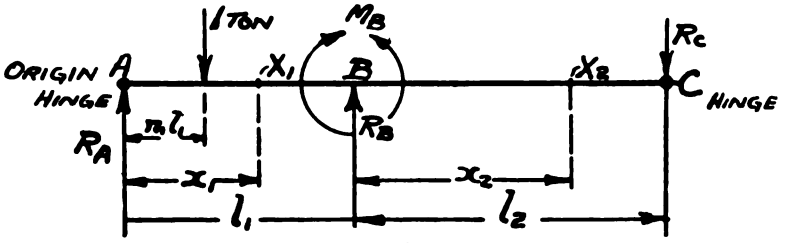

Fig. 6.

(*a*) Let unit load be in the first bay at a distance $n_1 l_1$ from the origin, the left-hand support A.

Fixing Moments	M_{AI}	M_{BI}	M_{CI}
Value	0	$\dfrac{n_1 l_1^2(1-n_1^2)}{2(l_1+l_2)}$	0
Reactions	R_{AI} ↑	R_{BI} ↑	R_{CI} ↓
Value and direction of acting.	$(1-n_1)-\left[\dfrac{n_1 l_1(1-n_1^2)}{2(l_1+l_2)}\right]$	$n_1+\dfrac{n_1 l_1(1-n_1^2)}{2(l_1+l_2)}+\dfrac{n_1 l_1^2(1-n_1^2)}{2l_2(l_1+l_2)}$	$\dfrac{n_1 l_1^2(1-n_1^2)}{2l_2(l_1+l_2)}$

The moment at some section X_1 in the first bay distant x_1 from A and between the load and B is

$$EI\frac{d^2y_1}{dx_1^2} = -R_{AI}x_1 + (x_1 - n_1 l_1)$$

$$EIy_1 = -R_{AI}\frac{x_1^3}{6} + \frac{(x_1 - n_1 l_1)^3}{6} + Px_1 + Q \quad . \quad . \quad . \quad . \quad (14)$$

when $x_1 = 0$: $y = 0$. $\therefore Q = 0$.
,, $x_1 = l_1$: $y = 0$.

Substituting in equation (14), and solving

$$P = \frac{l_1^2}{6}[R_{AI} - (1 - n_1)^3]$$

For sections in the first span between A and the load, i.e. for $x_1 < n_1 l_1$

$$EIy_1 = -R_{AI}\frac{x_1^3}{6} + \frac{l_1^2}{6}[R_{AI} - (1 - n_1)^3]x_1 \quad . \quad . \quad . \quad . \quad (15)$$

For sections in the first span between the load and B, i.e. for $x_1 > n_1 l_1$,

$$EIy_1 = -R_{AI}\frac{x_1^3}{6} + \frac{l_1^2}{6}[R_{AI} - (1 - n_1)^3]x_1 + \frac{(x_1 - n_1 l_1)^3}{6} \quad . \quad . \quad (16)$$

For Sections X_2 in the second span distant x_2 from B

$$EIy_2 = -\frac{R_{AI}(l_1 + x_2)^3}{6} + \frac{l_1^2}{6}[R_{AI} - (1 - n_1)^3](l_1 + x_2)$$
$$+ \frac{(l_1 + x_2 - n_1 l_1)^3}{6} - R_B\frac{x_2^3}{6} \quad . \quad . \quad . \quad . \quad . \quad (17)$$

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(*b*) Let the unit load be in the second bay at a distance $n_2 l_2$ from B (*Fig.* 7).

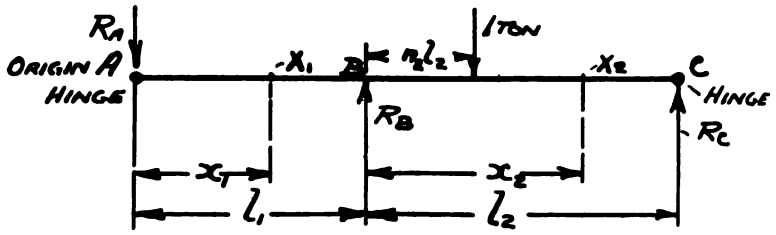

Fig. 7.

Fixing Moments.	M_{A2}	M_{B2}	M_{C2}
Value	0	$\dfrac{n_2 l_2^2(2-3n_2+n_2^2)}{2(l_1+l_2)}$	0
Reactions Value and direction	R_{A2} $\dfrac{n_2 l_2^2(2-3n_2+n_2^2)}{2l_1(l_1+l_2)}$ ↓	R_{B2} $(1-n_2)$ $+\dfrac{n_2 l_2^2(2-3n_2+n_2^2)}{2l_1(l_1+l_2)}$ $+\dfrac{n_2 l_2(2-3n_2+n_2^2)}{2(l_1+l_2)}$ ↑	R_{C2} $n_2-\dfrac{n_2 l_2(2-3n_2+n_2^2)}{2(l_1+l_2)}$ ↑

The moment at any section X_1 in the first bay distant x_1 from A (the origin) is

$$EI\frac{d^2y_1}{dx_1} = R_{A2}x_1$$

$$\therefore\ EIy_1 = R_{A2}\frac{x_1^3}{6} + P_1x_1 + Q_1. \quad . \quad . \quad . \quad . \quad . \quad (18)$$

$$x_1 = 0 : y_1 = 0 \quad \therefore\ Q_1 = 0$$
$$x_1 = l_1 : y_1 = 0$$

and substituting in equation (18) and solving

$$P_1 = -\frac{R_{A2}l_1^2}{6}.$$

The deflection at sections in the first span distant x_1 from A is therefore

$$EIy_1 = R_{A2}\frac{x_1^3}{6}. - \frac{R_{A2}l_1^2}{6}.x_1 \quad . \quad . \quad . \quad . \quad . \quad (19)$$

The deflection at sections in the second span between B and the load for which $x_2 < n_2 l_2$

$$EIy_2 = \frac{R_{A2}(l_1+x_2)^3}{6} - \frac{R_{B2}(x_2)^3}{6} - \frac{R_{A2}l_1^2}{6}(l_1+x_2) \quad . \quad . \quad . \quad (20)$$

The deflections at sections in the second span between the load and C, i.e. for which $x_2 > n_2 l_2$.

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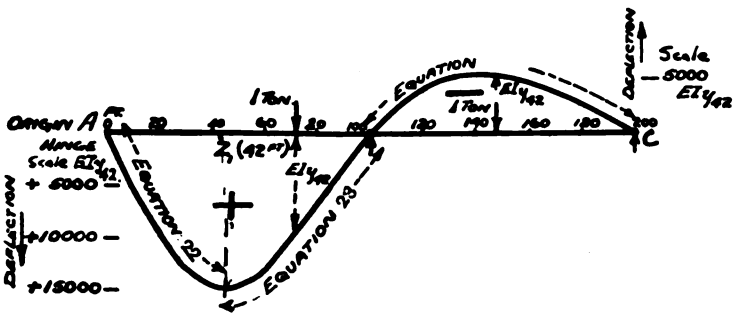
$$EIy_2 = \frac{R_{A2}(l_1 + x_2)^3}{6} - \frac{R_{B2}x_2^3}{6} + \frac{(x_2 - n_2l_2)^3}{6} - \frac{R_{A2}l_1^2}{6}(l_1 + x_2) \quad . \quad . \quad (21)$$

If the spans are of the same length l, the equations can easily be adjusted by substituting l for l_1 and l_2.

For a number of concentrated dead loads in the span, the deflection of the beam is easily solved by the straightforward method.

Unit Influence Lines of Deflection as a unit load passes over the span.

(*a*) For a section Z_1 in the first bay distance z_1 from the origin A (*Fig.* 8) the load will be at a varying distance n_1l_1 from A, whilst in the first bay.



UNIT INFLUENCE LINE OF DEFLECTION × EI FOR SECTION Z_1 42 FT. FROM THE ORIGIN.

Fig. 8.

Let the load be between A and Z_1, i.e. $n_1l_1 < z_1$ then adjusting equation (16),

$$EIy_{z1} = -R_{A1}\frac{z_1^3}{6} + \frac{l_1^2}{6}[R_{A1} - (1 - n_1)^3]z_1 + \frac{(z_1 - n_1l_1)^3}{6} \quad . \quad (22)$$

Let the load be between Z_1 and B, i.e. $n_1l_1 > z_1$, then adjusting equation (18),

$$EIy_{z1} = -R_{A1}\frac{z_1^3}{6} + \frac{l_1^2}{6}[R_{A1} - (1 - n_1)^3]z_1 \quad . \quad . \quad . \quad . \quad (23)$$

Let the load be in the second bay, then adjusting equation (19).

$$EIy_{z1} = R_{A2}\frac{z_1^3}{6} - \frac{R_{A2}l_1^2}{6} \cdot z_1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (24)$$

(*b*) For a section Z_2 in the second bay distant z_2 from B (*Fig.* 9), the load will be at a varying distance n_1l_1 from A whilst in the first bay and n_2l_2 from B whilst in the second.

Let the load be in the first bay, then adjusting equation (17),

$$EIy_{z2} = -\frac{R_{A1}(l_1 + z_2)^3}{6} + \frac{l_1^2}{6}[R_{A1} - (1 - n_1)^3](l_1 + z_2)$$
$$+ \frac{(l_1 + z_2 - n_1l_1)^3}{6} - R_{B1}\frac{z_2^3}{6} \quad . \quad . \quad . \quad . \quad (25)$$

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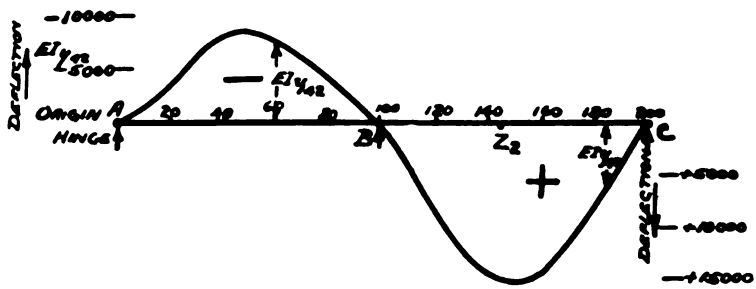
Let the load be in the second bay and between B and Z_2, i.e. $z_2 > n_2 l_2$, then adjusting equation (21)

$$EIy_{z2} = \frac{R_{A2}(l_1 + z_2)^3}{6} - \frac{R_{B2}z_2^3}{6} + \frac{(z_2 - n_2 l_2)^3}{6} - \frac{R_{A2}l_1^2}{6}(l_1 + z_2) \quad . \quad . \quad (26)$$

Let the load be in the second bay and between Z_2 and C, i.e. $z_2 < n_2 l_2$: then adjusting equation (20),

$$EIy_{z2} = \frac{R_{A2}(l_1 + z_2)^3}{6} - \frac{R_{B2}z_2^3}{6} - \frac{R_{A2}l_1^2}{6}(l_1 + z_2) \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$



UNIT INFLUENCE LINE OF DEFLECTION = EI
FOR SECTION Z_2 142 FT FROM THE ORIGIN

Fig. 9.

If a number of concentrated loads cross the span then $EIy_{z1} = \Sigma Wy$ as explained for a simple beam. The area of the unit influence deflection diagram above the length of a uniformly distributed load times the rate of loading is again the deflection for the given section when a uniformly distributed load crosses the span.

The previous method holds good for any number of spans: the working being obviously simplified for spans of the same length.

From the theorem of three moments a general expression can be found for the fixing moments at the supports for the unit load in any particular bay. The reactions and their direction can be found by a little thought. Always let the left-hand support be the origin and work from this. Then in the equation for EIy, the constant of integration Q will always be zero, and the value of the other constant P will always be found from the consideration that $y = 0$ (all supports same level), when x equal to the length of the first span.

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New Method of Panel Construction.

A NEW system of concrete panel construction applicable to housing is being used for the erection of houses in Perth by the Panelcrete Construction Co. The system has been evolved by Messrs. Smart, Stewart & Mitchell, of Perth, the architects to the Company, and the accommodation comprises living-room, 2 bedrooms, kitchenette, bathroom, etc., with hot and cold water supply.

The whole of the external walls are of hollow reinforced vertical concrete panels set in a slotted concrete foundation, while the interior slab partitions are also of special design. Instead of using compo board and straps as in steel house construction, the walls, ceilings and partitions are plastered throughout, and the roof is covered with red roofing tiles. The erection and completion of the house, including painting, fencing and laying-out of ground, etc., has been carried out within five weeks from the time the walls were commenced.

It is claimed for the new system, a patent of which has been applied for, that it combines the rapidity of erection possessed by the steel house together with the lasting qualities of the usual brick and roughcast structure. All the units are standardised, the wall panels, windows, and doorways being interchangeable, thus permitting of variations in plan and design. The system is adaptable to one or two-story houses. The whole of the work has been carried out under normal conditions with an ordinary staff of workmen working an eight-hour day and paid the standard rate of wages; no overtime has been put in on the work. We understand the Company are proceeding with the erection of another six cottages on the same site.

New System of Concrete House Construction.

The walls and partitions are fire and sound resisting, and there are two non-conducting air spaces in the outer walls. The outside walls are of cream white-washed rough cast, with white woodwork and red-tiled roof. The cost of these houses is about £500 each, varying according to the nature of the site and local conditions.

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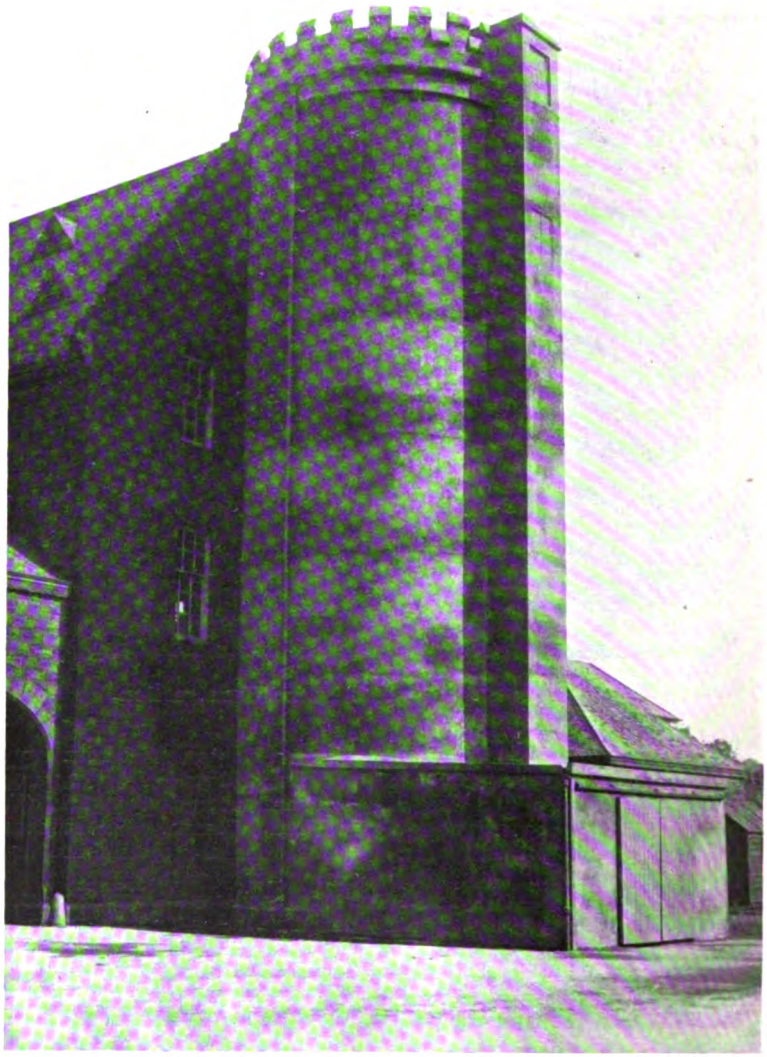

Reinforced Concrete Silo at Dumfries. (*See p.* 573.)

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Reinforced Concrete Silo at Dumfries.

BIG strides have been made in this country with the reinforced concrete silo—a type of building introduced by our American cousins—and the illustration on p. 572 shows one recently erected in Scotland. Mr. James Flett, clerk of works to the Crichton Royal Institution at Dumfries, is responsible for the general lay-out and architectural design, to which Messrs. The Trussed Concrete Steel Co., Ltd., adapted their "Hy-Rib" system of silo construction.

The silo belongs to that class of building which, if not handled with care from the point of view of appearance, will readily become an eyesore, and bearing in mind the restrictions of utility and economy the treatment of the silo illustrated is far from displeasing, largely due to the crenellated parapet creating the "finished" effect so often lost when a roof is left exposed like the top of some huge pillar box.

The main cylinder has a diameter of 16 ft. and is 50 ft. high measured from the base slab (2 ft. 6 in. below ground level) to the collar beneath the parapet, and is capable of storing 242 tons of ensilage when full. At the rear, and attached to the wall in a similar manner to the rainwater pipe seen in the photograph, is a feed pipe which enters the silo through a dormer in the conical roof behind the parapet while the chute at the front protects a series of doors 1 ft. 9 in. wide by 4 ft. high in the main wall and is lighted by two steel-framed windows near the top. The doors (eight in number) are conveniently reached by means of a steel-rung ladder fixed to the main wall alongside them inside the chute, and the bottom of this ladder rests on the floor of the lean-to shed. Opposite the highest window of the chute is a door giving access to another dormer in the roof, and from this across to the dormer accommodating the feed-pipe is a gangway suspended by steel hangers from the roof slab. This gangway enables speedy attention to be given to the pipe in the event of its being clogged during filling operations.

A rather unusual feature is the lean-to shed adjoining the foot of silo, which besides being extremely useful to the owners also serves as a trap for the offensive odours given off by ensilage and is a welcome innovation. The lowest door of the silo is central with the sliding doors of this lean-to, and the chute, being built off the roof, appears from the interior much in the form of a chimney flue.

With the exception of a concrete frame supporting the sliding doors of the shed and reinforced with steel bars, the whole of the walls and roofs are reinforced with "Hy-Rib"—a rigid steel ribbed lathing the use of which eliminates close boarded shuttering. The "Hy-Rib" sheets in the silo wall (fixed with the ribs horizontal) are curved to the necessary sweep and stiffened vertically by means of ⅜-in. diameter rods at regular intervals, which are spliced at the bottom to rods cast in the concrete foundation. Portland cement rendering composed of 1 part Portland cement to 3 parts sand was worked up on the "Hy-Rib" internally and externally in a series of coats to form the wall slab, which decreases in thickness towards the top. The wall is 4 in. thick from the ground level to a height of 13 ft., where a set-back of half-an-inch allows a 3½-in. slab to be maintained up to 33 ft., and at this level a further set-back reduces the size to 3 in., for the remainder.

In order to obtain a perfectly flush surface outside the changes in the thickness of the wall are made internally, with the result that the capacity of the silo is increased. The latter remark may seem, at first glance, hardly worthy of mention, but as a matter of fact, the setting back of the wall on the inside added nearly 113 cubic feet to the volume of the silo. This was certainly worth considering. All work in connection with the wall was accomplished from a scaffold platform erected around the structure for the fixers and plasterers.

The conical roof is principally formed of a 4 in. Portland cement concrete slab laid on "Hy-Rib" sheets, which are fixed in this instance with the ribs radiating from the peak and strengthened with the cranked rods tying in the cheeks of the dormers. The walls of chute and lean-to are 3 in. thick, with the steel lathing vertically in the former and horizontally in the latter with additional rods introduced transversely to the direction of ribs in the sheets.

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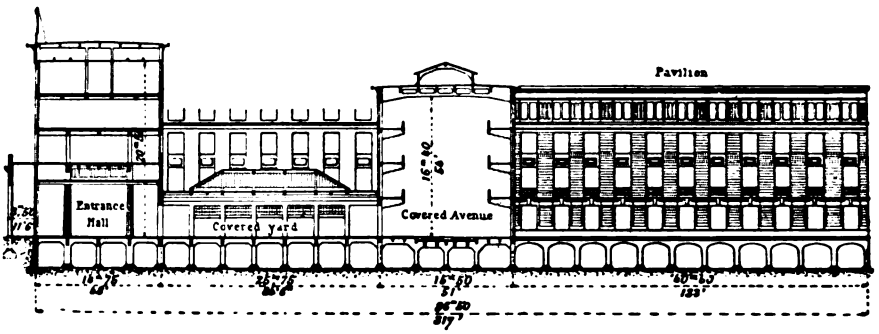
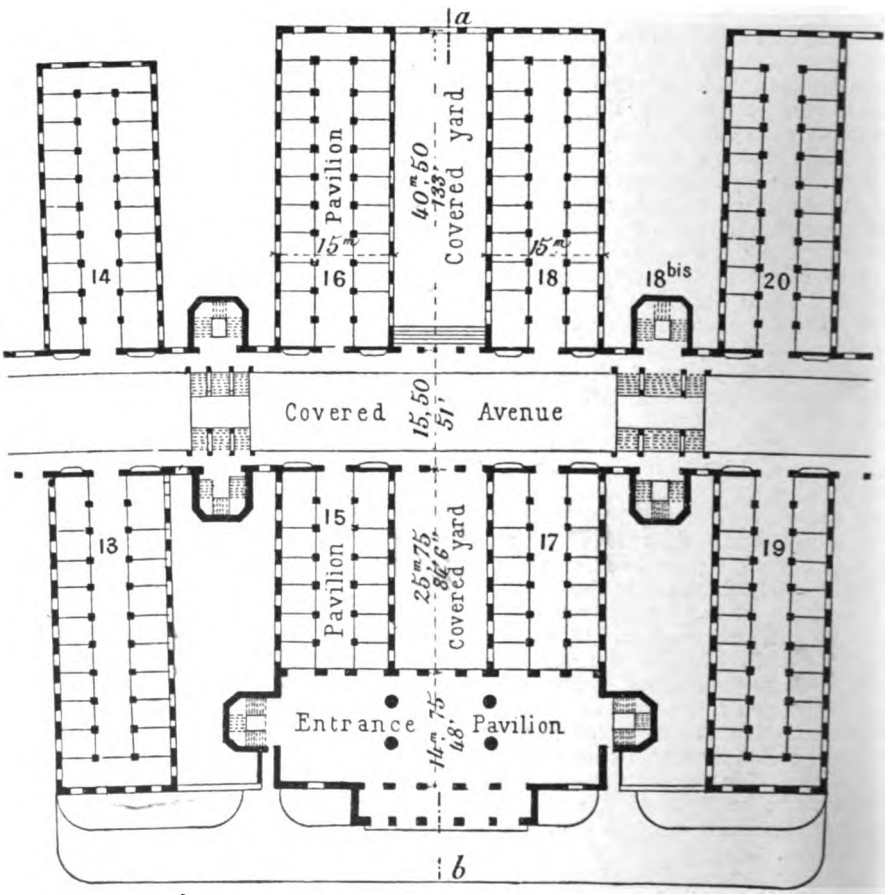


Transverse Section on Line a—b.



Part Plan.

Reinforced Concrete Buildings at Lyons Fair. (*See p.* 575.)

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Reinforced Concrete Buildings at Lyons Fair.

STARTED in 1916 with a view to competing with the famous Leipzig Fair in Germany, considerable progress has been made with the erection of the permanent buildings at Lyons Fair. First held in timber buildings in 1918, it was decided that these structures should be replaced with reinforced concrete, and so far twenty-two pavilions, containing 2,000 "stands," have been completed. This is about half the ultimate size of the exhibition, which it is proposed shall contain a total of about 4,000 stands.

The site covers 24 acres on the bank of the Rhone, and on account of the liability of the district to floods, the whole of the buildings have been built on reinforced concrete columns in turn resting on reinforced concrete piles. The piling is on the "Simplex" system; that is, a steel tube of about 16 in. diameter is driven and filled with concrete in

Lyons Fair: A Glimpse of the Completed Buildings.

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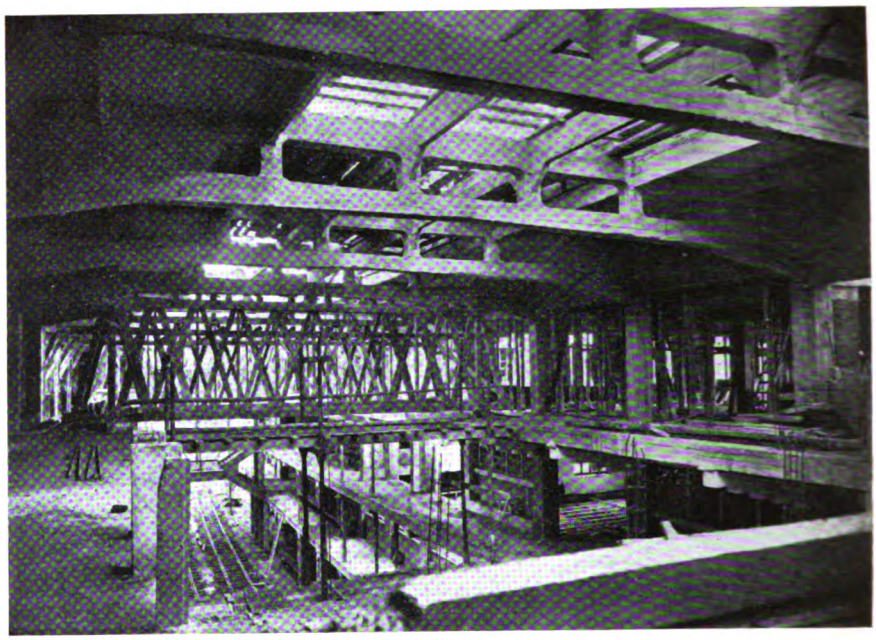
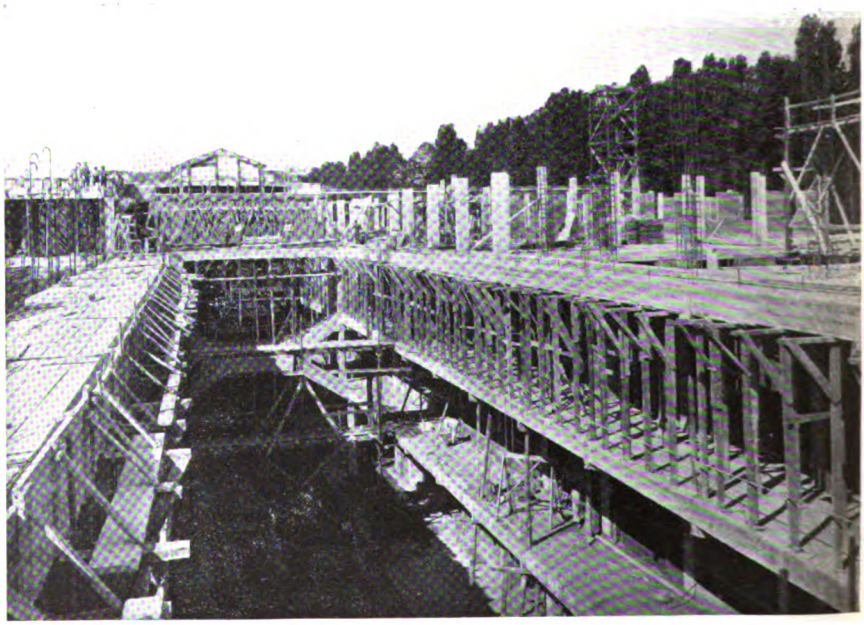


Roof of Covered Avenue.



View along Avenue during Construction of Stands.

Reinforced Concrete Buildings at Lyons Fair.

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sections about 36 in. in length, each deposit of concrete being rammed as it is placed, and the steel tube raised as each section is rammed. Rafts are laid over sets of two, three, or four of the piles to act as bases for the columns supporting the building. The buildings containing the stands, all three stories in height, are of reinforced concrete frame construction, the walls being filled in with slag concrete, and faced with cast stone. Passageways projecting on cantilevers into the avenues give access to the stands on the upper floors.

The main hall in the Entrance Pavilion extends through the ground and first floors, and as columns were not desired the weight of the upper portion is carried by the external walls. The second floor is therefore suspended by columns from the roof beams. In order to support this weight the roof beams are 7 ft. 4 in. deep, although the width is comparatively small. The roofing of the covered avenue is carried on hollow reinforced concrete beams of 51 ft. span (see p. 578).

The architect is M. Meysson, and the engineer, M. Chalumeau. So far a total of 47 million francs has been spent on the constructional work.

Cement Mortars at High Temperatures.

THE behaviour of standard mortar test-specimens under the action of high temperatures, which is important in the design of reinforced concrete chimneys, has been the object of a research by Dr. Nitzsche, whose results are given in a recent issue of *Zement*.

The test-specimens were prepared according to the German rules and using, (1) Portland cement, (2) the same Portland cement but with an addition of ground bituminous materials as in the manufacture of Antiaqua cement, (3) slag cement, and (4) slag cement with a bituminous admixture. The mortars were kept one day in the forms, six days in water, and afterwards in air. At the age of nine months the specimens were heated in an electric oven and afterwards dried to constant weight at 112° F. The results for Portland cement are given in the table.

The cements used were somewhat below the standard of fineness of grinding and for that reason the proportional tensile strengths at six months' age are given in the table.—W.S.G.

PORTLAND CEMENT.

	Per cent. loss in weight.	Tensile strength lbs./in.²	Heated 120 hours to	Ref.
Tensile strength of air-stored mortar at six months.		368		
	1·87	217	302° F.	I
	2·18	199	392° F.	II
	1·83	104	482° F.	III
	2·13	128	572° F.	IV

PORTLAND CEMENT AND BITUMINOUS ADMIXTURE.

Tensile strength of air-stored mortar at six months.		304		
	1·73	146	302° F.	1
	2·05	128	392° F.	2
	2·28	161	572° F.	3
	2·81	81	752° F.	4

Concrete Sleepers.

FOR experimental purposes one of the leading railway groups is negotiating for the purchase of concrete sleepers, which will be laid on a stretch of the main line out of London. If the experiment proves successful, and concrete sleepers are adopted generally, it will mean the ultimate replacement of a hundred million wooden sleepers now lying on the railway tracks of Great Britain. The new sleeper consists of two blocks of concrete joined together by a tie-rod of mild steel. Each block, which is 24 in. long and 7 in. thick, holds three plugs arranged to take the screws of the chair carrying the rail. In order to hold the ballast the underside of the blocks is slightly concave; and the edge is curved to suit the stroke of the "beater" (the hammer used by the platelayer) and so facilitate packing. The chair rests on a layer of felt, and the inventor claims that travel over such sleepers has proved to be less noisy than over any other kind of track.

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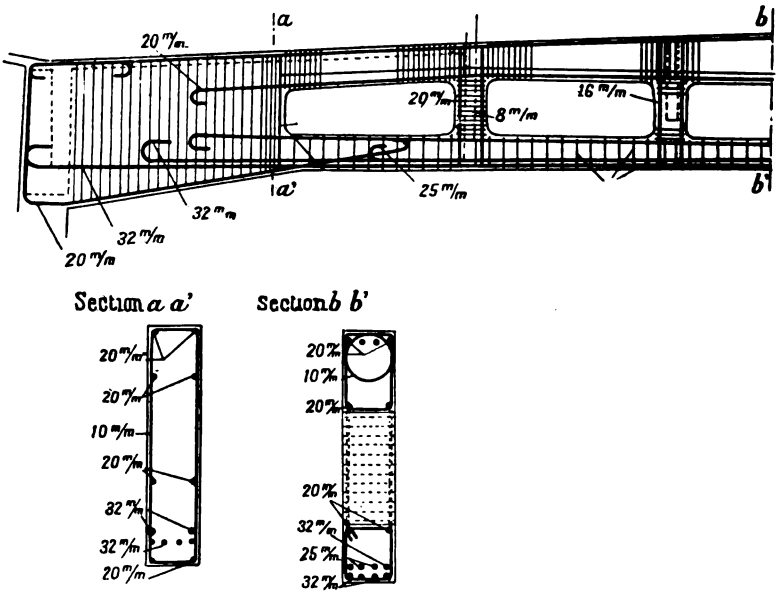
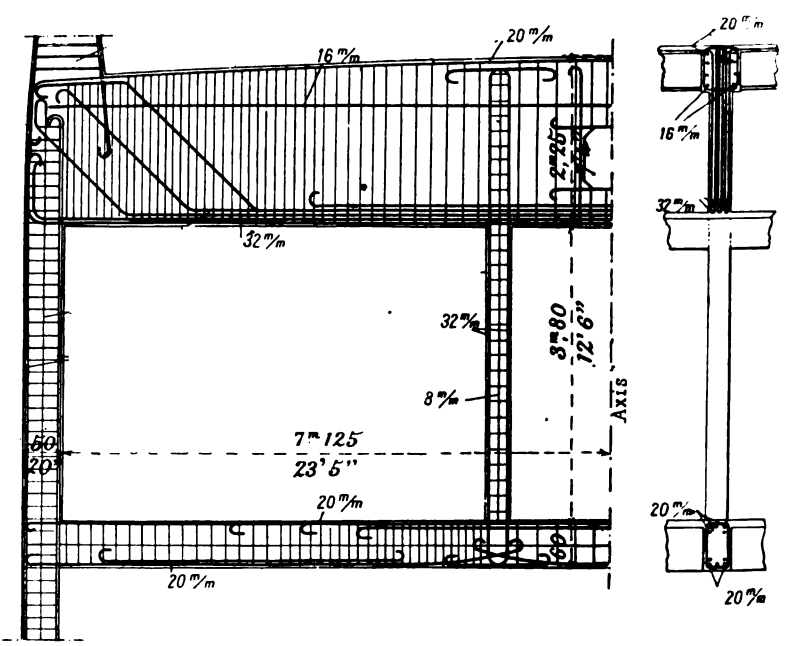


Details of Hollow Roof Beam in Covered Avenue.



Suspension of Second Floor to Roof Beams.

Reinforced Concrete Buildings at Lyons Fair. (*See p.* 575.)

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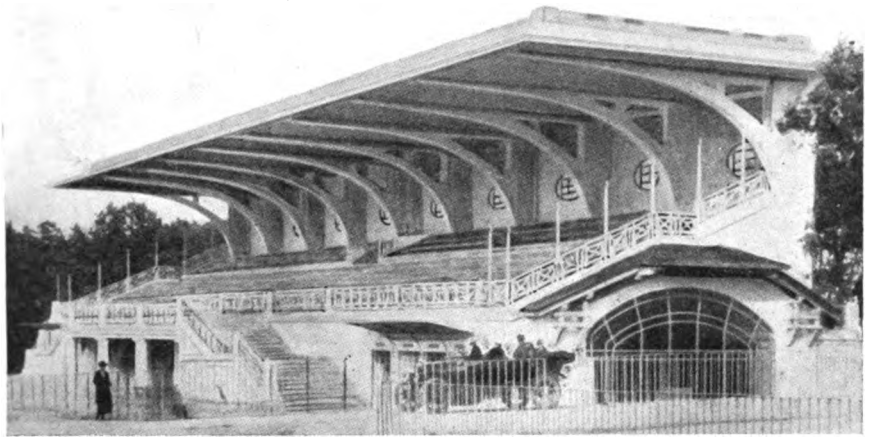
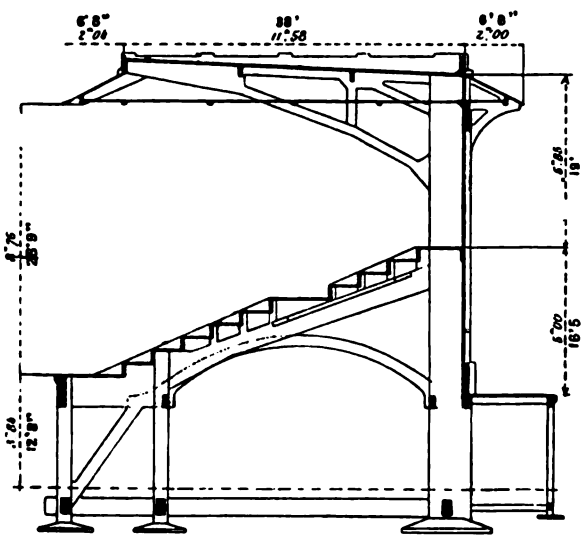


Reinforced Concrete Grand Stand at Vincennes.

Reinforced Concrete Grand Stand at Vincennes.

On account of the obstruction of the columns of the brick stand which formerly did duty at the "Bois de Vincennes" race course, near Paris, a new stand in reinforced concrete has just been completed. The stand is 43 ft. long by 49 ft. deep, and the roof has an overhang, supported by cantilevers, of more than 45 ft. The general construction is shown in our illustrations.

The cantilevers are spaced at 21 ft. 4 in. centres, and support a reinforced concrete roof slab 2⅜ in. thick. A live load of 120 lbs. per sq. ft. was assumed for all parts of the structure accessible to the public, and an extra load of 12 lbs. per sq. ft. for the roof to allow for snow. A wind-pressure of 30 lbs. per sq. ft. on the vertical surfaces is assumed. Accommodation is provided for 1,500 seats.



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Design of Formwork for Reinforced Concrete Construction.

By A. E. Wynn, B.Sc., A.M.Am.Soc.C.E.

X.—FORMS FOR RIB FLOORS AND STRUCTURAL STEEL FIREPROOFING.

RIB FLOORS.

THIS type of construction is commonly used when the live load is light and a flat ceiling unobstructed by beams is desired. The span of the slab may vary from 8 to 30 ft., but is generally about 20 ft. The ribs are formed by fillers, which may be made of hollow clay tile, steel, or gypsum, and occasionally of wood.

The filler is generally left in, although metal tiles are made which can be stripped and used again ; wood forms are never left in. Whether it is economical or not to remove the fillers depends on the first cost and the number of times they can be used. Removable tiles are much heavier and hence cost more, and when the cost of stripping is added it is usually cheaper to leave the tiles in place unless they can be used many times over.

With metal tiles, unless the plastered ceiling is suspended, it is usual to lay the plaster lath direct on the forms, in which case the depth of the lath must be allowed for in calculating the height of the forms. When no plastered ceiling is required the tiles are removed, and in this case wood forms are sometimes built.

Hollow clay tiles are 10 in. to 12 in. wide and the ribs usually 4 in. wide, so that the tiles are placed 14 in. to 16 in. on centre. Metal and gypsum tiles are about 20 in. wide at the base, and the ribs 4 in. to 5 in. wide, so they are placed 24 in. to 25 in. on centre. Wooden fillers can be made any desired width and height, but are usually not over 30 in. wide and 12 in. high.

The spacing of the ribs mainly governs the type of form used. With clay tile fillers a " closed deck " is nearly always used ; that is, the floor is completely sheathed over. For other fillers an " open deck " is used, that is, the sheathing is only under the ribs. With ribs 16 in. on centre if an open deck is used with 2 in. by 8 in.'s. under the ribs the same amount of timber is required as with a closed deck. The 2 in. timber is required as with a closed deck. The 2 in. timber will cost considerably more than the 1 in., and will cost more to place, as the planks have to be spaced carefully and fillers will be required to form the T to the concrete beam. Also, with an open deck there is some possibility of losing concrete at the bottom of the ribs unless tile spacers are used. Instead of 2 in. by 8 in. plank, 2 in. by 6 in. may be used, but this gives only 1 in. bearing for the tile, and the planks have to be spaced very accurately. There is, however, more salvage value with 2 in. timber than with 1 in., and in some cases, as, for instance, if it is known that the

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planks can be used again several times, it may be advisable to use an open deck with clay tile.

With ribs 24 in. on centre an open deck will save one-third of the sheathing, although the plank will cost more per cubic foot. The load in this case is concentrated at the ribs, so that is where the support should be; a closed deck would not distribute the load over 24 in.

Closed Deck. (*Fig.* 53).—This will be very similar to forms for beam and girder construction. Ledgers will be required to carry the joists; they may be single or double timbers, and should be cleated to the posts. If double they should be nailed together, and both timbers should not be spliced over the same post.

The floor sheathing can be loose boards or panels, depending on the number of times they can be used. The edges resting on the beam sides should be bevelled and have ¼ in. clearance. The joists should be lightly nailed to the ledger here and there to keep them in place. The posts will be on wedges as before. The post cleats are preferably nailed to the posts before erection, so saving labour in erection.

Beam sides and connections to columns will be the same as for beam and girder construction, but since the depth below the slab is generally small a single board is often deep enough, so that no battens are required.

Beam forms are erected first, then posts and ledgers either separately or together. They must be set plumb and braced in both directions. To set to correct height a straight-edge can be used running on the beam side ledgers. Then the joists and sheathing are placed and the wedges are gone over to bring the floor level.

To strip, the beam sides are removed first as a unit and the beam reshored. The wedges under the posts are loosened and the posts and ledgers are removed together, putting in a temporary shore under the joists or leaving in a centre ledger until the joists are removed. If double ledgers are used, the nails at the splices—which should not be driven home—should be drawn before stripping.

If the joists are in short lengths, after loosening the wedges they can be knocked over and removed with the sheathing without removing the ledgers, and these can be wedged up again and left in longer. The slab forms should be left in as long as beam bottoms, and when stripped one or two lines of posts about 10 ft. apart should be replaced with the attached ledgers until the concrete has been poured for at least 28 days.

Since the width of the sheathing between beams will be much greater than in beam and girder construction, boards should be left out here and there to allow for swelling, or if panels are used there should be clearance between each panel.

Open Deck. (*Fig.* 54).—Two-inch planks take the place of 1 in. sheathing, in other respects the construction is much the same for open decks as for closed decks. The planks should preferably be one length, but may be spliced if necessary with cleats on the underside.

Ledgers and posts can be erected separately or together in bents of three or four posts, holding them temporarily at the top with a joist. The filler will nearly always stop a few inches from the beam sides to

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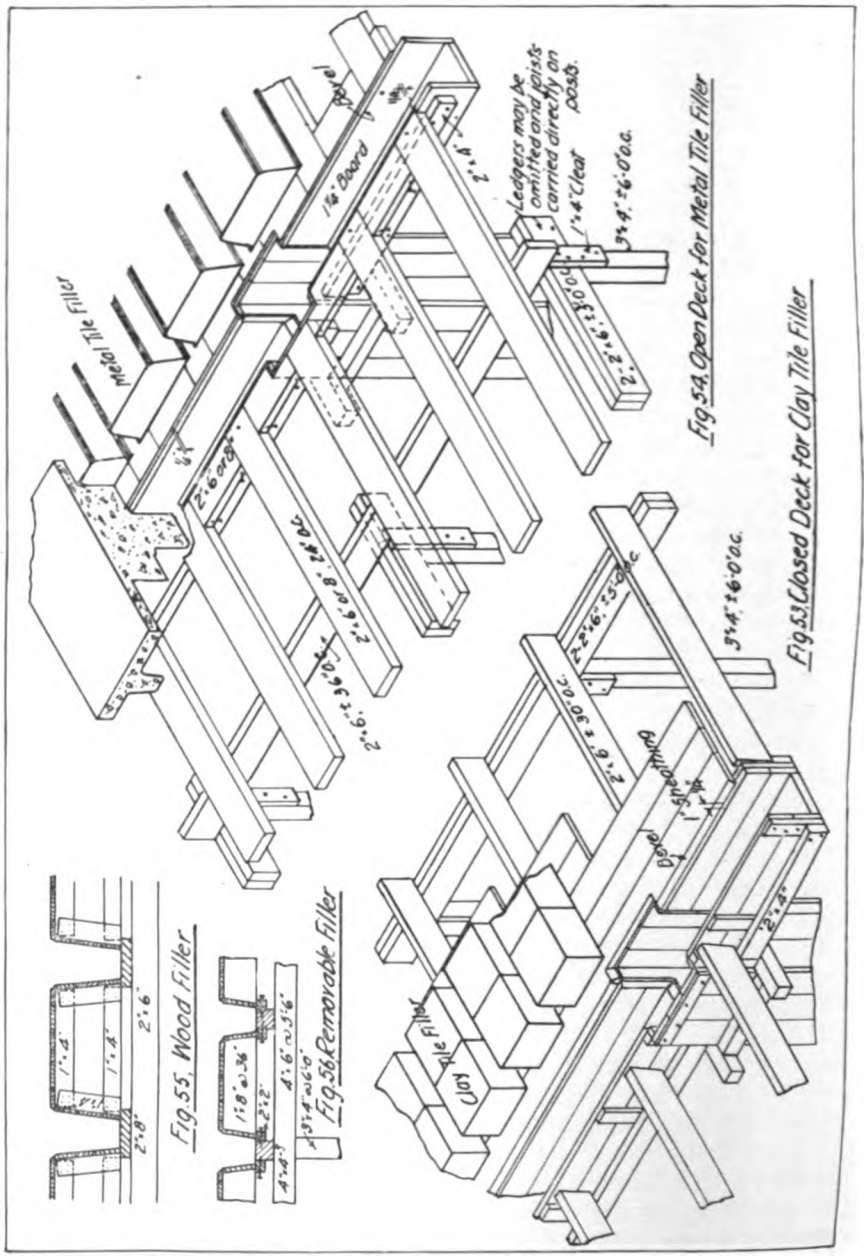

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provide a solid concrete Tee of the total depth of the slab. To provide for this Tee, short pieces of plank, an inch or so wider than the width of the Tee, have to be inserted between the rib bottoms, and nailed to them with heads left projecting. The short pieces should all be ready, cut to exact length, and can be used as spacers for the planks, placing a plank and short piece alternately. If the short pieces are placed after the planks there may be some trouble and time wasted in fitting them in. One or two heavy nails will hold the plank to the beam ledger and a nail at every other joist will hold the joists in place.

The plank can be carried on the beam sides, as in *Fig.* 53; that is, resting on the top with ¼ in. clearance. A better construction is shown in *Fig.* 54, where the plank butts against the sides and is ¼ in. higher, with the beam sides bevelled. It is easier to bevel the beam sides than the plank. The beam sides are held at the bottom with spikes, as they are never very deep. The sides are often not over 8 in., so one board can be used. Joists generally cover three spans, or are about 16 ft. long.

A method sometimes used (though it is not so good) is to run the joists under the centre of and parallel to the planks. The plank and joist are nailed together to form a unit, and the plank rests on top of the beam sides, being about 1 in. longer each end than the joist.

The above remarks on stripping apply also to the open deck. The short filler planks should have the nails drawn before stripping, to free the planks. When reshoring the ledgers should be placed at right angles to the ribs, and the posts must be blocked up from the floor. Ledgers may be omitted, and the joists carried directly on the posts. About the same amount of timber will be required, but there will be twice as many posts to place, plumb, and wedge. There is an advantage in that the joists only have to be levelled, instead of both joists and ledgers.

Wood Fillers. (*Fig.* 55).—Instead of steel removable tiles, or when the spacing of the tiles is too great for the standard widths, wood forms are often used. They may be economical on a small job, where only used once or twice, but if they are to be used several times steel tiles are nearly always cheaper as there are no repairs and they have almost full salvage value.

The wood form is made of 1 in. stock nailed to 1 in. frames, forming panels 8 ft. to 10 ft. long. The sides forming the ribs and the ends should slope at least 1 in. from top to bottom in order to facilitate stripping. The frames should be about 36 in. apart and made of 1 in. by 4 in., or 1 in. by 6 in., or they can be made solid with battens. A few diagonal braces are useful to prevent buckling. The frame is supported in the same way as for metal tile fillers. There may be some difficulty in stripping the forms, as they are liable to stick unless made with a hinge at the top of the frame, which adds greatly to the labour cost, and is not worth while unless the forms are carried as more or less permanent plant. It will help stripping to have the two end top boards narrow and loose.

A method of building the forms so that the filler can be stripped without disturbing the supports is shown in *Fig.* 56. The filler joists in this case

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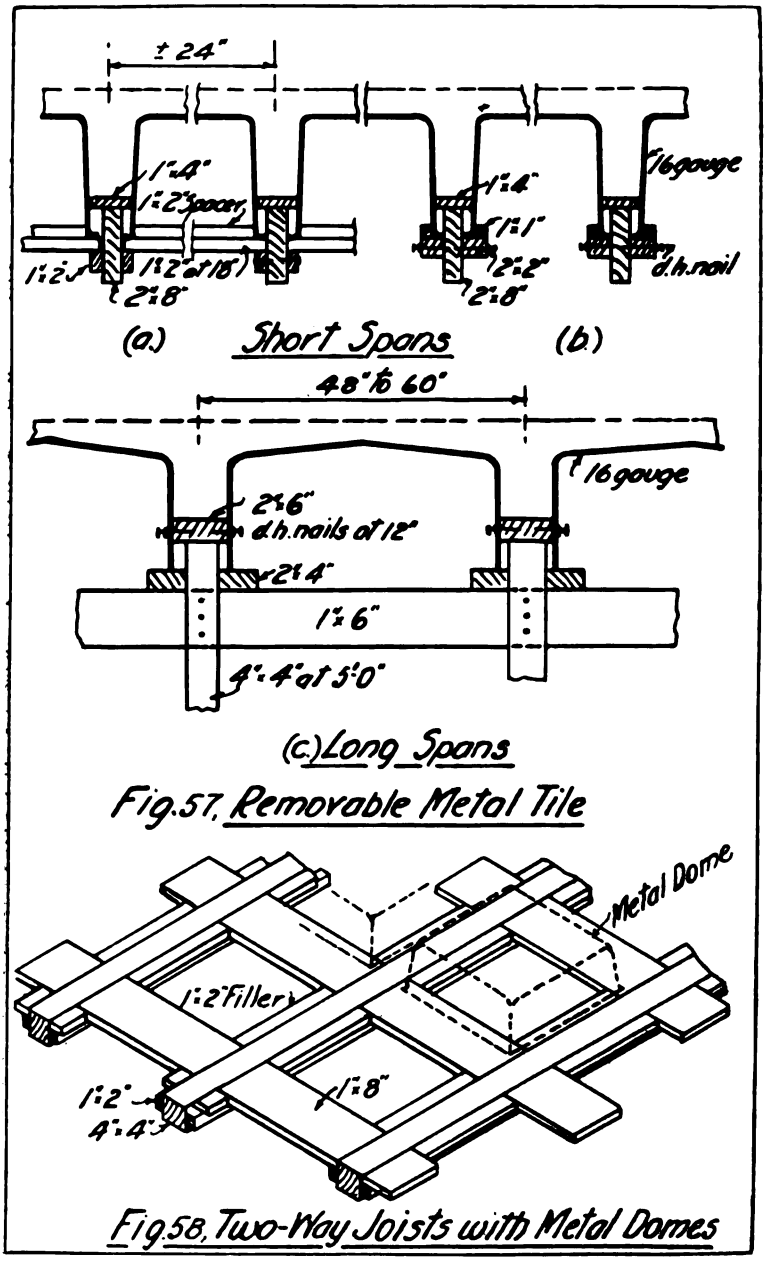

Fig. 57, Removable Metal Tile

Fig. 58, Two-Way Joists with Metal Domes

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are single boards of the depth required; if a stock size will not suit the next widest board can be used and the ends sized down. The joist bottoms are 4 in. by 4 in. carried on 4 in. by 6 in. ledgers. On the sides of the 4 in. by 4 in.'s are nailed 2 in. by 2 in. strips—using double-headed nails—which support the fillers. Removing these strips enables the fillers to be removed without disturbing the joist supports. The length of the fillers must be less than the distance apart of the ledgers, say about 3 ft. to 4 ft.

Removable Metal Tiles.—These are economical for high buildings or very large areas, when they can be used at least four times over, or when for architectural reasons a beam ceiling is desired.

Fig. 57 (*a*) and (*b*) shows two methods of building the forms when the joists are close together, say about 24 in. So that the tiles will be economical for use on different jobs, they are made deep enough to suit several different depths of joist. The joist support is a 1 in. by 4 in., or whatever width of joist is specified, nailed to a 2 in. plank on edge. The tiles overlap the joist bottom and at (*a*) are carried on 1 in. by 2 in. about 18 in. apart supported by 1 in. by 2 in. ledgers nailed to the plank. Spacers of 1 in. by 2 in. will prevent the tiles spreading at the bottom. To strip, the 1 in. by 2 in. supports are knocked off the ledgers and the tiles are sprung out. The tiles are made in about 3 ft. lengths. At (*b*) the ledgers are 2 in. by 2 in., and 1 in. by 1 in. ribbons are used to hold the bottom of the tiles. In either case the plank can be supported directly by posts or by ledgers to reduce the number of posts.

For longer spans between joists the method shown in *Fig.* 57 (*c*) can be used. The joist support is a 2 in. by 6 in. nailed to the top of 4 in. by 4 in. posts. The tiles are carried on 2 in. by 4 in.'s supported on continuous 1 in. by 6 in.'s nailed to the posts. The tiles are also nailed —through holes supplied for that purpose—to the joist bottom. Double-headed nails are preferable whenever they are to be pulled before stripping.

Usually one-and-a-half or two floors of steel tiles are used, so that there is always one floor fully formed beneath the floor being poured. The floor that is stripped should be reshored. The metal tiles can be removed in two to three days for the short spans and four to six days for the long spans.

Two-Way Joist Construction.—In simple joist construction the joists run in one direction, being carried by two beams or walls. If it is desired to distribute the slab load more equally a "two-way" construction is used; that is, the joists run in two directions at right angles, and are supported on four beams or walls. The fillers then, instead of being continuous rows, will be square units, or "domes" as they are generally called.

This construction is more common with steel fillers than with hollow tile because closed ends are required, while the ordinary hollow tiles have open ends. If hollow tiles are used the construction is the same as the closed deck already described. The load to be carried will be slightly greater, as there will be more concrete, so the supports must be a little closer.

Metal tiles, or "domes," about 20 in. square at the base are supported

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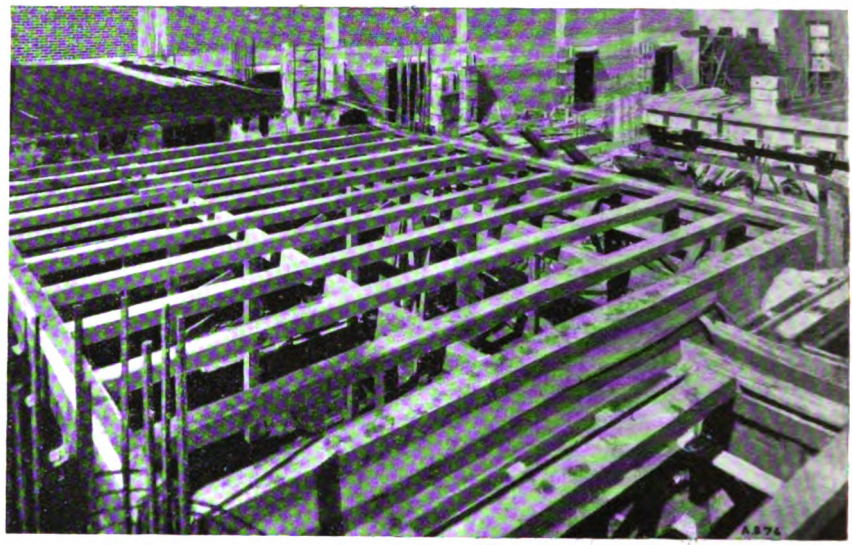

[By permission of The Trussed Concrete Steel Co.

FIG. 60.—FORMS FOR REMOVABLE METAL TILE.

[By permission of The Trussed Concrete Steel Co.

FIG. 61.—FORMS FOR WIDE-SPAN REMOVABLE METAL TILE.

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on open deck forms, but the construction shown in *Fig.* 54 is not easily adapted for domes because the joist bottoms running in one direction must support the bottoms in the direction at right angles, and this is not easily done with 2 in. plank.

Instead, the construction shown in *Fig.* 58 can be used. The joist bottoms in one direction are 4 in. by 4 in.'s. To each side of the 4 in. by 4 in., and 1 in. down from the top, is nailed a 1 in. by 2 in. ledger. This ledger carries 1 in. by 8 in.'s at the same distance apart as the 4 in. by 4 in.'s, forming the bottoms for the joists in the opposite direction. On top of the ledgers, in between the 1 in. by 8 in.'s, are nailed 1 in. by 2 in. fillers, carrying the domes. The 4 in. by 4 in.'s are supported as in *Fig.* 54, but the supports must be closer together because of the greater dead load.

An alternative to this method is to substitute for the 1 in. by 8 in. joist bottom a 16-gauge iron plate the width of the joist and supported by the dome flanges. In this case the ledgers are 2 in. by 2 in., nailed flush with the top of the 4 in. by 4 in., and are continuous, no filler strip being required.

With either construction the domes can be removed without disturbing the supports. *Figs.* 59, 60, 61, show construction of forms for fixed and removable metal tile fillers manufactured by Trussed Concrete Steel Co.

Estimating Cost.—Closed deck forms will require about $\frac{1}{4}$ cu. ft. of timber per sq. ft. of floor area, and open deck forms about $\frac{1}{5}$ cu. ft. per sq. ft. for average conditions. Open deck construction therefore requires about 20 per cent. less timber than the closed deck. These amounts include posts, bracing, waste, etc. The labour cost is about

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FIG. 59.—OPEN DECK FORMS FROM ABOVE, SHOWING "FLORE-TYLE" AND "HY-RIB" LATH.

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the same for either method, since with the former there is less timber, but the work has to be done more accurately, and with the latter there is more timber to handle but erecting it is simpler. There will, however, be more salvage of material with the heavier timber of the open deck.

A full set of forms should be estimated for one floor or a floor and a half, and these can be used again on upper floors.

To frame and erect forms for 100 sq. ft. of floor will require about 5 hours carpenters' time and 5 hours labourers' time, and to strip them about ½ hour carpenters' time and 2 hours labourers' time.

Cost of forms for 100 sq. ft. of floor (open or closed deck),

5½ hrs. carpenter @ .....
7 ,, labourer @ .....

For closed deck, timber required = 100/4 = 25 cu. ft.
,, open ,, ,, ,, = 100/5 = 20 ,, ,,

Cleaning and hoisting will each require about 1 hour labourers' time.

Beams can be estimated as in beam and girder construction, also reshoring.

The cost of wood fillers should be estimated separately from the forms and be compared with the cost of tile or metal fillers. They will require about ⅛ cu. ft. of timber per sq. ft. of floor area, and it will require about 5 hours carpenters' time and 2 hours labourers' time to make up sufficient fillers for 100 sq. ft. of floor.

The cost of forms for wood fillers and removable metal tile is about the same as for the open deck in both material and labour.

Forms for two-way joists will cost about 10 per cent. more in material and labour per sq. ft. of floor area than the open deck.

STRUCTURAL STEEL FIREPROOFING.

The forms for supporting the concrete fireproofing around a structural steel frame (*Fig.* 62) are built much simpler and lighter than for reinforced concrete frames. Cinders as aggregate instead of stone are used to cut down the dead weight of the structure, forming a concrete weighing only about 100 lbs. per cu. ft., or about two-thirds the weight of stone concrete. Little or no live load need be allowed for in designing the forms, as wheeling of concrete is done on runways supported directly by the steel beams. As there is practically always a suspended ceiling beneath the floor and other exposed concrete is plastered, the lines and appearance of the concrete are not so important, so that strength in bending rather than allowable deflection will govern the form design.

The floor forms are always suspended from the steel beams, eliminating all wood posts, so that the forms do not have to carry the floor above as in reinforced concrete construction. Dressed boards, nominally 1 in. thick, being the cheapest timber to buy, are used practically throughout. Often only one width of board is necessary, being mainly governed by the depth required for the joists. The usual widths are

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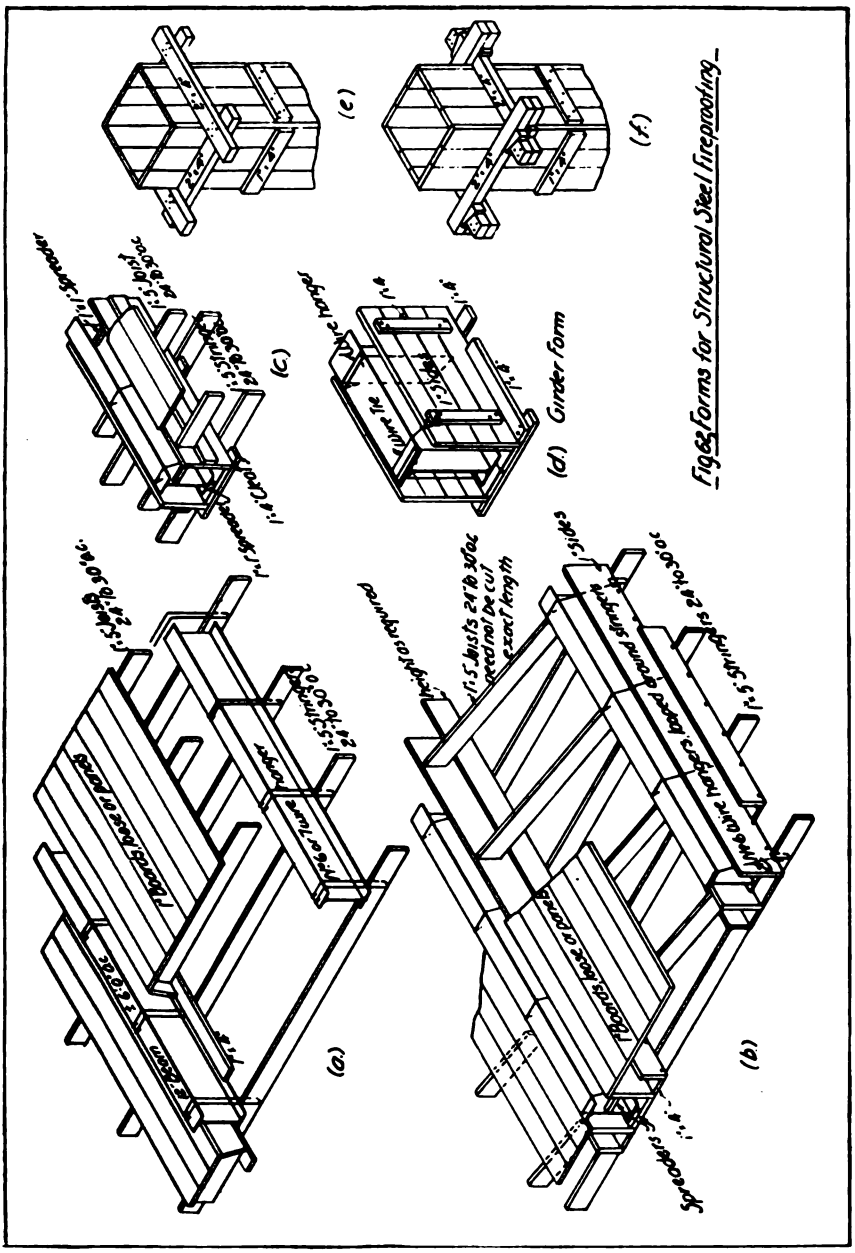

Fig. 62. Forms for Structural Steel Fireproofing.

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4 in., 5 in. or 6 in., dressed about ¼ in., with square edges. Having only one size to deal with greatly simplifies the timber list and reduces the labour. Hangers consist of heavy bright wire number 6 or 7.

The work of erection should be subdivided into placing hangers, beam forms, stringers, etc. The general method is to carry the beam and slab forms by continuous stringers hung from the top flange of the beams.

The floor slab is usually 4 in. thick, of cinder concrete, and will weigh about 35 lbs. per sq. ft. The span between beams is generally between 6 ft. and 8 ft. The safe carrying capacity of 1 in. joists, depending on their strength in bending for various spans and spacing, is as follows:—

				lbs. per sq. ft.
For 5 ft. span,	1 in. by 4 in.	at 30 in.	on centre will carry	25
		at 24 in.	,, ,, ,, ,,	35
,, 6 ft. ,,	1 in. by 5 in.	at 30 in.	,, ,, ,, ,,	30
		at 24 in.	,, ,, ,, ,,	40
	1 in. by 6 in.	at 30 in.	,, ,, ,, ,,	45
		at 24 in.	,, ,, ,, ,,	55
,, 7 ft. ,,	1 in. by 6 in.	at 30 in.	,, ,, ,, ,,	30
		at 24 in.	,, ,, ,, ,,	40
,, 8 ft. ,,	1 in. by 6 in.	at 24 in.	,, ,, ,, ,,	30
		at 18 in.	,, ,, ,, ,,	35

The span of the joists is the span between supports, not centre to centre of the beams.

The stringers can be the same size and spaced similarly to the joists. The slab boards are either laid loose or built into panels. The beam sides, where possible, consist of one width of board, but for deep beams two or more boards must be nailed together with battens.

Beam bottoms are also 1 in. thick and generally single boards. Girder bottoms are usually wider, and require two boards with battens. Wire hangers should be number 6 or 7 bright wire, not black wire. The sequence of erection is wire hangers, stringers, beam and girder forms, joists and slab panels.

At (*a*) *Fig.* 62 is shown the construction of the forms when the underside of the beams is not fireproofed. It is best suited for shallow beams, 10 in. or less in depth, so that a 5 in. or 6 in. joist can be used. This system can be conveniently built in panels 6 ft. to 8 ft. long, nailing the joists, beam sides, and slab boards together. The panels are then carried by 1 in. by 4 in.'s resting loosely on the stringers. The beam sides should be battered for easy stripping.

Wire hangers should be cut to required length and bent before placing. The stringers are pulled up tight against the lower flange of the beam, and the ends of the hanger hammered tight up around the top flange. In this case both legs of the hanger are shown carried over the top flange, but the more usual construction is shown at (*b*). The length of the hooks under the top flange should be about 4 in.

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It is more usual to encase the entire beam in concrete, and the best construction for the forms is shown at (*b*), *Fig.* 62. In this case the joists are carried on L-shape supports, resting loosely on the stringers; the horizontal leg may be 4 in. to 6 in. and both legs are nailed together. The advantage of this method of joist support is that the span of the joist is reduced, and the joists need not be cut to the exact distance between beam sides as is necessary when they are carried by ledgers nailed to the sides. By placing the joists on a skew their lengths can vary an inch or so and they can be wedged against the sides, thus keeping them in place without nailing and also holding the sides from bulging. The slab boards may be loose or made into panels, in which case the two outside boards should be loose.

The usual construction for the hangers is to loop one end over the stringer and the other end over the top flange, hammering it well under the flange. The hangers pass between the bottom and side forms. Spreaders 1 in. by 1 in. should be used at intervals to space the forms from the bottom and sides of the steel beam.

Stripping in either method of construction consists merely in cutting the hangers and lowering the forms.

If the beam is so deep that two ordinary width boards are not sufficient to form the joists and vertical leg of the joist support, the joists are then carried as at (*c*) on a ledger nailed to the sides. The girder form shown at (*d*) consists of side and bottom panels made up with 1 in. by 4 in. cleats, 24 in. to 30 in. apart. The wire hangers pass through and under the bottom cleats and the legs are vertical, and are

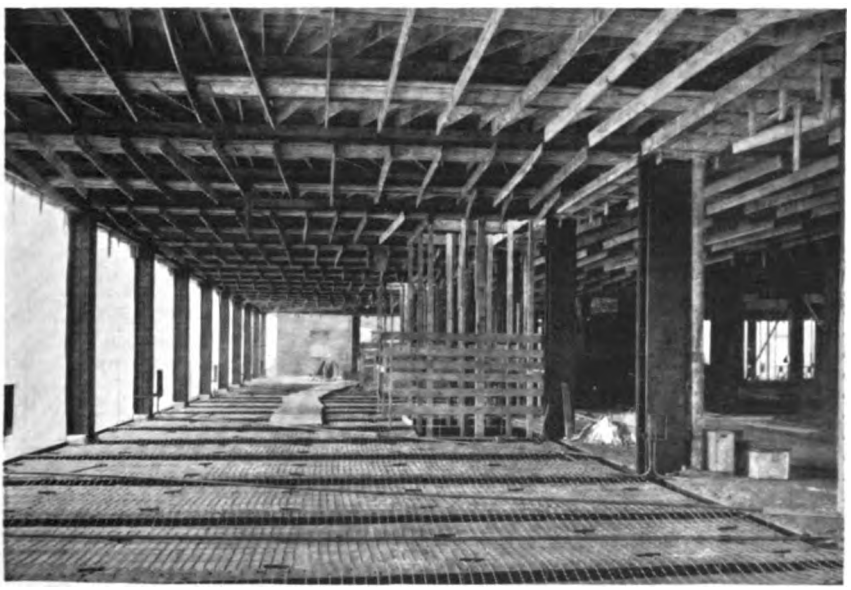

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FIG. 63.—SUSPENDED FORMS FOR STRUCTURAL STEEL FIREPROOFING, EQUITABLE BUILDING, NEW YORK.

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bent around the top flange. The holes in the cleats should not be in a line parallel to the sides, but one hole should be near one edge and the other hole near the opposite edge; that is, the wire passes diagonally under the cleat. The top of the girder is held from bulging by wire ties around the side cleats, or it can be braced back diagonally to the beam sides. If the joists are carried on girder sides the construction will be as at (*c*.)

Column forms are built much lighter than in reinforced concrete construction, as the pressure will be much less. Generally the sides are panels of 1 in. boards cleated together every 24 in. to 30 in. The yokes are loose and can be 2 in. by 4 in. nailed together as at (*e*), or wedged together as at (*f*) *Fig.* 62. The latter method requires more labour cutting and fitting the wedges and stop blocks, but is much easier to strip. Patent adjustable column clamps are often used in place of wood clamps.

Connection of beams to girders and girders to columns can be made as previously described, using 1 in. stock instead of 2 in.

Stripping of beams and girders can be done in two to three days, and slabs in three to seven days.

If stone concrete is used instead of cinder concrete the same methods of construction can be used, although since the dead load will be greater all the forms must be correspondingly heavier, and instead of wires, bolts will be required hooked on the lower or top flange with washer and nut under the beam bottom cleat.

Estimating Cost.—From $\frac{1}{8}$ to $\frac{1}{4}$ of a cu. ft. of timber should be allowed per sq. ft. of total contact area, including slab, beams, and columns; this will include waste. In high buildings sufficient timber should be provided to form from two to three whole floors.

The labour cost will vary greatly with the experience of the contractor in this class of work and with the size of the job and speed required. It will vary from as high as the cost of forms for reinforced concrete beam and girder construction, as given in Chapter IX, to about 25 or 30 per cent. less than this. In America this work is largely handled by firms who specialise in fireproofing structural steel as sub-contractors, doing no other part of the work. Since these firms have experienced men who do nothing but build forms for structural steel fireproofing they can obtain much lower costs than the general contractor, the average cost being about 30 per cent. less than that given in Chapter IX.

(*To be continued.*)

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Thames ballast	,,	10	6
Broken brick (¾ in.)	,,	11	6
Best British Portland Cement	per ton 58s. to	63	6
"Ferrocrete" Rapid-Hardening Portland Cement	delivered London 10s. per ton extra		
"Super-Cement"	per ton 88s. to	93	0
"Lightning" Brand Aluminous Cement	,, 108s. to	113	6
Ciment Fondu	130/- ex stores		

BOARDING FOR SHUTTERING—		Sawn. s. d.	Wrot. s. d.
1 in.	per square	23 6	27 6
1¼ in.	,,	29 6	33 6
1½ in.	,,	35 6	41 0

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in.	from £23 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £23 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	11	0
3/16 in. to ½ in. Rounds	,,	11	3
⅜ in. Rounds	,,	11	6
¼ in. Rounds	,,	12	6

Breeze Slabs per yd. super: 2 in., 1/11; 2½ in., 2/4; 3 in., 2/9; 4 in., 3/6.

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 : 2 : 4—		s.	d.
Do. do. in foundation	per yard cube	48	6
Do. do. in columns	,, ,,	53	6
Do. do. in beams	,, ,,	53	6
Do. do. in floor slabs 4 in. thick	per yard super	5	8
Do. do. in floor slabs 6 in. thick	,, ,,	8	5
Do. do. in floor slabs 9 in. thick	,, ,,	12	3
Do. do. in walls 6 in. thick	,, ,,	8	8

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—		s.	d.
From ¼ in. to ⅝ in.	per cwt.	23	0
,, 3/16 in. to ½ in.	,,	22	0
,, ⅝ in. to 2⅞ in.	,,	21	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1¼*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2½*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.* (per bend per cwt.).

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.* (per bend per cwt.).

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	,, ,,	1	1½
Do. do. to sides and soffits of beams, average 9 in. by 12 in.	,, ,,	1	1½
Do. do. as last, in narrow widths	,, ,,	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Carpenters working on old shuttering, 1/10½; Labourers on building works, 1/4½; Men on mixers and hoists, 1/5½; Bar-benders, 1/5½.

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Prospective New Concrete Work.

ACLE.—*Bridge.*—A proposal to build a bridge at Acle, Norwich, is being considered by the Norfolk Highways Committee.

AYRSHIRE. — *Road.* — The Ayrshire Northern District Committee proposes to construct a by-pass road at Stevenston, at a cost of £21,458.

BALLYBOGHILL. — *Concrete Bridge.* — The Dublin C.C. proposes to replace the old bridge with a reinforced concrete structure.

BALROTHERY, Co. DUBLIN.—*Concrete Cottages.*—The R.D.C. proposes to build 52 concrete cottages, estimated to cost between £270 and £300 each.

BARMOUTH.—*Piling.*—A big piling scheme is being considered for the reconstruction of the damaged promenade.

BEDWORTH.—*Houses.*—The Foleshill R.D.C. has applied for a loan of £16,600 for the erection of 40 houses at Bulkington Lane.

BIRTLEY. — *Road.* — The Chester-le-Street R.D.C.'s roads scheme includes the construction of a road from Bog Lane to Pelton-Ouston Road. The estimated cost is £15,502.

BLAYDON.—*Houses.*—The U.D.C. proposes to erect 60 houses by direct labour.

BRIDLINGTON.—*Roads.*—The Corporation has applied to the M.H. for sanction to borrow £3,250 for the provision of roads on the Postill estate.

BRISTOL.—*Houses.*—The T.C. has applied for a loan of £29,490 for the erection of 62 houses.

BURTON-ON-TRENT. — *Houses.* — The Corporation has purchased land for the erection of 100 houses.

CARDIFF.—*Bridges.*—The Corporation proposes to spend £30,000 on the construction of bridges over Lake Road East and Lake Road West.

CHADDERTON.—*Houses.*—The R.D.C. is considering a scheme for the erection of 70 houses.

CHESTER.—*Houses.*—The Corporation's housing scheme includes the erection of 103 houses on the Heath Lane site.

DALKEITH.—*Houses.*—The T.C. has asked for a loan of £29,000 for housing purposes.

DARLINGTON.—*Houses.*—The Corporation Streets Committee recommends the erection of 50 houses.

DERBY. — *Houses.* — The Corporation proposes to erect 250 concrete and steel houses.

DONINGTON. — *Bridge.* — The Donington High Bridge is to be reconstructed in reinforced concrete.

EASTBOURNE.—*Houses.*—The T.C. has purchased 30 acres of land for housing purposes.

EDINBURGH.—*Footbridge.*—A scheme for the erection of a footbridge over the railway at Tay Street is being considered.

GLOUCESTER.—*Houses.*—The R.D.C. has applied for permission to erect 50 houses.

GREAT YARMOUTH.—*Houses.*—The Corporation proposes to erect 100 houses.

GREENOCK. — *Bridge.* — The Streets Committee is considering a proposal to erect a bridge across Crescent Street.

GRIMSBY.—*Bridge.*—The Corporation proposes to spend £51,000 on replacing the old bridge at Alexandra Dock with a bridge across Corporation Road.

HALIFAX.—*Houses.*—The T.C. proposes to spend £91,100 on the erection of 218 houses.

HEMSWORTH.—*Houses.*—The R.D.C. proposes to build 500 houses in the district.

HORSFORTH.—*Houses.*—The U.D.C. is considering a scheme for the erection of 100 houses.

HOUGHTON.—*Houses.*—The R.D.C. has applied for sanction to erect 200 houses.

HOVE.—*Houses.*—The Corporation proposes to build 96 houses on the Old Shoreham Road estate.

LEEDSTOWN. — *Concrete Tank.* — The Helston R.D.C.'s scheme for the water supply for the village of Leedstown includes the construction of a reinforced concrete tank to hold 3,000 gallons.

LLANFAIRFECHAN — *Viaduct.* — The U.D.C. is considering a scheme for the construction of a viaduct to avoid the Penyclip Hill. The estimated cost is £100,000.

LOWESTOFT. — *Houses.* — A loan of £13,175 is being asked for by the T.C. for the erection of 31 houses.

MAIDSTONE. — *Road.* — The T.C. has applied for a loan of £4,000 to purchase land in connection with a proposal to construct a road from King Street to Church Street.

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MANCHESTER.—*Houses.*—Of the 600 houses which are to be erected at Moston, 300 are to be built by direct labour.

MARGATE.—*Sea Wall, etc.*—The Corporation has applied for a loan for the construction of a sea wall and undercliff promenade, which is estimated to cost £36,000.

MERIONETH.—*Bridge.*—The C.C. proposes to erect a bridge at Bont Newydd in place of the existing structure.

MORECAMBE.—*Houses.*—The Corporation has decided to spend £80,000 on housing.

NEWMARKET.—*Houses.*—The R.D.C. proposes to build 200 houses during the next three years.

PERSHORE, WORCS.—*Wall.*—A concrete wall, estimated to cost £300, is to be erected at Pershore where the River Avon reaches the roadway.

PORTSMOUTH.—*Houses.*—The Corporation proposes to erect 72 houses at Stamshaw, estimated to cost £38,000.

PRESTON. — *Houses.* — The T.C. proposes to erect 100 houses.

RETFORD.—*Houses.*—A Committee has been appointed to inquire into a proposed scheme for the erection of 74 houses.

ROTHERHAM.—*Road.*—The R.D.C. has applied for a loan for the construction of a road from Vale Road to the Doncaster-Tinsley main road. The estimated cost is £5,430.

RUSHDEN.—*Houses.*—The U.D.C. proposes to purchase land in Irchester Road for the erection of 46 houses.

SADDLEWORTH. — *Houses.* — A scheme for the erection of 65 houses is being considered.

SALISBURY. —*Houses.* — The Corporation is considering a scheme for the erection of 50 houses.

SOKEHOLME.—*Concrete Bridge.*—The Skegby R.D.C. proposes to construct a reinforced concrete bridge over Sokeholme ford, at an estimated cost of £850.

STANLEY, YORKS.—*Bridge.*—A scheme is in hand for the construction of a bridge over the River Aire at Bottomboat.

SUNDERLAND. — *Retaining Wall.*—The Corporation is considering a scheme for the construction of a wall at the cliffs at Roker, estimated to cost £18,000.

SWANSEA.—*Houses.*—Application has been made to the M.H. for a loan for the erection of 100 houses.

TADCASTER.—*Houses.*—The R.D.C. has applied for a loan of £138,690 for the erection of 306 houses.

TAVISTOCK. — *Houses.* — The R.D.C. proposes to build 100 houses.

TRURO.—*Houses.*—The C.C. proposes to erect 33 houses.

TURRIFF.—*Reservoir.*—The T.C. propose to construct a reservoir to hold a month's supply of water.

WALTHAMSTOW (LONDON).—*Houses.*—The U.D.C. has asked for a loan of £97,658 18*s.* for the construction of 198 houses by direct labour.

WEALDSTONE.—*Houses.*—The U.D.C. has asked for a loan of £59,950 for housing schemes.

WISBECH ST. MARY.—*Houses.*—The R.D.C. has applied for sanction to build 62 houses.

Tenders Accepted.

BARNSLEY.—The T.C. has accepted the tender of Messrs. Hodkin & Jones, Ltd., at £560, for concrete work at the pumping station.

CATHERINGTON, HANTS.—*Reserviors.*—The R.D.C. has accepted the tender of Messrs. F. Bevis, Ltd., at £17,454, for the construction of two reinforced concrete reservoirs, a pumping station, and laying cast-iron water mains.

GLASGOW.—The Corporation has accepted the following tenders of Mr. A. W. Paterson, of Kelvingrove, for the formation of paddling pools and sand pits in the public parks :—Kelvingrove Park, £458 9*s.* 5*d.*; Alexandra Park, £405 2*s.* 8*d.*; Richmond Park, £448 8*s.* 8*d.* and Elder Park, £472 8*s.* 4*d.*

GLASGOW.—*Bridge.*—The Corporation has accepted the tender of Messrs. R. McAlpine & Sons, at £24,630, for the construction of a reinforced concrete bridge over the Duntocher Burn.

GLASGOW. — *Footpath.* — The Corporation has accepted the tender of Mr. J. Drysdale, at £518, for the construction of a concrete footpath at Ruthenglen Road.

HUNDON, SUFFOLK.—*Bridge.*—The W. Suffolk C.C. has accepted the tender of Mr. H. Debenham, of Tye Green, Glemsford, at £335 7*s.* 3*d.* for the reconstruction

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of Babel Green Bridge in concrete, and the construction of mass concrete retaining and abutment walls.

KILKENNY.—*Water Tank.*—The C.C. has accepted the tender of Messrs. Connolly & Cullen, Kilkenny, at £199, for the construction of a concrete water tank at the Sanatorium.

LEEDS.—*Bridge.*—The Corporation has accepted the tender of Messrs. H. Arnold & Son, for the construction of a reinforced concrete bridge across Farnley Beck.

MACCLESFIELD.—*Concrete Road.*—The Highway Committee has accepted the tender of Messrs. Connolly, Roycroft, Ltd., Urmston, at £455 14*s.* 7*d.* for the construction of approximately 1,250 super yards of concrete road.

MARKET HARBOROUGH.—*Bridge.*—The R.D.C. has accepted the tender of Messrs. K. Holst & Co., 1 Victoria Street, S.W.1., at £865, for the construction of a reinforced concrete bridge over the canal.

MEVAGISSEY, CORNWALL.—*Wall, Etc.*—The St. Austell R.D.C. has accepted the tender of Mr. J. H. Lobb, Mevagissey, at £1,218 10*s.* 3*d.*, for the construction of reinforced concrete walls and road over Portmeelon Beach.

OSSETT.—*Footpaths.*—The Corporation has accepted the tender of Messrs. Luther, Ingham & Co., Ltd., Ossett, for concreting footpaths in Northfield Road, Lowood Avenue, Wilson's Avenue, Manor Road and Teall Street.

TAUNTON.—*Houses.*—The T.C. has accepted the tender of Messrs. F. & E. Small, Taunton, at £18,228, for the erection of 42 concrete houses.

THORPE HOUSE, YORKS.—*Retaining Wall.*—The North Riding C.C. has accepted the tender of Messrs. J. Busfield, Ltd., Leeds, for the construction of a concrete retaining wall.

New Companies Registered.

CONCRETE HOUSES, LTD. (207895). Registered August 15. Adelphi Terrace House, W.C.2. To acquire patents and rights relating to construction of buildings from E. W. Roberts. Nominal capital, £10,000. Permanent directors: Oscar Faber, 37, Duke Street, W.; F. E. Williams, F.R.I.B.A.; and A. Cox, F.R.I.B.A., 34, Henrietta Street, W.C.2; and two others.

CONCRETE MANUFACTURING CO. (BATTERSEA), LTD. (207931). Registered August 17. 317, High Holborn, W.C.1. To take over the business of concrete manufacturers carried on by the Concrete Manufacturing Co., at Battersea. Nominal capital, £1,000. Directors: A. G. Stedman, 317, High Holborn; A. E. Scrutton, 317, High Holborn, W.C.1.

GOSS MOOR CONCRETE CO., LTD. (208274). Registered September 7. Glencoe, Roche, Cornwall. Manufacturers of concrete blocks, bricks, posts, etc. Nominal capital, £5,000. Directors: E. M. Jackson, Glencoe, Roche, Cornwall; H. F. Jackson, Glencoe, Roche, Cornwall; A. G. Stevens, 6, Leyborne Avenue, Ealing.

NIGERIAN PORTLAND CEMENT SYNDICATE, LTD. (207866). Registered August 13. 151, High Street, Hull. Manufacturers of cement, limes, etc. Nominal capital, £20. Directors: G. G. Wellsted and E. Dronke.

Recent Patent Applications.

227,837.—E. C. Eckel: Process of making cement and iron.

231,141.—P. J. F. Kestner: Refractory cement.

236,259.—Sir E. Airey: Construction of concrete buildings and pillars therefor.

236,278.—J. Temperley: Concrete mixers.

236,450.—Novocretes, Ltd., G. O. Cage and J. R. Garrow: Concrete blocks, slabs, panels, &c.

236,481.—C. Martin and R. W. Snowden Concrete block moulding machines.

Trade News.

Mr. H. G. Blakemore has taken temporary offices at No. 180, Piccadilly, W.1 (telephone, Gerrard 8263), where he is organising an agency business in connection with machinery, plant and materials of interest to contractors, builders, municipal engineers and quarry owners.

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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EDITORIAL NOTES.

A New Edition of the B.S.S. for Portland Cement.

THE successive editions of the British Standard Specification for Portland cement have the character of landmarks in the development of the cement industry, and the new edition—the fifth—just issued is more outstanding than any of its predecessors. The details of the Specification and the changes that have been made are set out and discussed on another page.

It is by comparing the standards of quality required in the first edition of the Specification issued in 1904 with those of the latest edition that a true picture is obtained of the great advances that have occurred in the technique of cement manufacture. Taking the 7-days sand test as the keynote of the Specification, the minimum requirement of to-day is nearly three times as great as that of twenty years ago, and 62½ per cent. in advance of the standard in the Specification issued only five years ago. Herein lies an explanation of the strides that have been made in the concrete industry. Concrete piles or concrete roads

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could not have been expected with cement of the 1905 standard of quality, and these and many other examples of concrete construction have only become possible because of the improvement in the quality of cement. The present position of the concrete industry has, in fact, only been achieved as a result of the development of the cement industry, and, if so much has been done in twenty years, what are the possibilities of the future?

The new Specification sets a higher standard of quality than any other national cement Specification, and this cannot fail to raise the prestige of British cement in the world markets. It is indeed a worthy position that has been attained by British cement manufacturers in being able to supply a product carrying with it a higher guarantee than any other country can offer when judged by its national Specification, and it is all the more creditable that the stringent requirements of the new Specification have not been forced upon British manufacturers, but rather have the latter led the way in demanding a Specification more in keeping with the cement being produced. The Committee responsible for the preparation of the Specification is representative of many interests, including engineers, contractors, cement experts, cement manufacturers, Government departments, and scientific societies. No particular section of the Committee predominates, and the Specification therefore commands confidence as meeting the needs of cement users without making unreasonable demands on cement manufacturers.

It has been common knowledge that the revision of the Specification is somewhat overdue; that is to say, the majority of the British cements on the market are much superior in quality to the requirements of the 1920 Specification, and a statement that a cement complied with the British Standard Specification did not convey much real information as to quality. The raising of the standard that has now taken place corrects this anomaly, and the new Specification more nearly represents the average quality of British cements.

The change in the strength requirements that has been made in the Specification is of considerable importance in connection with concrete design, because unless a special brand of cement had been specified the engineer until now could only assume in his calculations that the cement used would no more than comply with the low requirements of the Specification, although in most cases the strength of the cement was double the specified minimum. As a consequence, the improved quality of the average British cement was not fully taken advantage of because all cements were not up to the average, although complying with the Specification. The great advance in the minimum strength of the cement now gives the engineer a new basis on which to calculate, and there should be possibilities of economy in concrete design. The Committee are to be congratulated on the bold and striking changes that have been made in the Specification.

Education in Structural Engineering.

THERE is usually considerable difference of opinion among engineers as to the best form of education for the young men entering the profession; at one extreme we have the ultra-practical people who think college training gives the students swollen heads that render them unsuited to the exacting conditions of practical

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life, and at the other extreme we have the too theoretical people who underestimate the importance of practical considerations. In spite, however, of the cold water which the ultra-practical people throw upon theoretical knowledge—from much the same motives that moved the fox who had his tail cut off to harangue his brethren—the number of candidates for University degrees in engineering has grown considerably in recent years, and considerable progress must be taking place in the popular appreciation of technical education. In structural engineering sound technical knowledge—which should not be confused with advanced mathematical facility—is essential for those who wish to become designers. This is true of all forms of structural work and applies particularly to reinforced concrete in which practical development was delayed until further technical knowledge had been developed by experimental research and scientific investigation.

That the educational side of structural engineering should receive the attention of our educational bodies has been recognised in an interesting report which has recently been made by H.M. Inspectors to the Board of Education. The report is intended to provide the members of the Board with knowledge of the structural engineering industry and deals almost entirely with structural steelwork, although it refers occasionally to reinforced concrete; it does not deal with engineering schools of University rank, but rather with technical schools and polytechnics which devote most of their energies to young men already engaged in the industry either in the workshop or the drawing office. Educational institutions of this kind have for years been carrying on good work with but little encouragement from the education committees who run them or real appreciation from the students who attend them. It is frequently the case that the education committees do not know anything about engineering or education, and the students are often so deficient in general education that they are unable to benefit from the technical instruction given to them.

The report in question states that "Although practical designing experience in a teacher is most desirable, it should never be forgotten that the most important function of the teaching is to give the theory which cannot generally be learnt as part of the work of a firm's drawing office." With this we are in complete agreement, and would advise students to concentrate upon obtaining a clear idea of scientific method and the basic principles of structural design rather than to attempt to get at once to something which is of direct application to their drawing-office work. Many students fail through attending for several years a number of isolated classes from few of which they obtain much nourishment because they have never given sufficient time to the general principles of mechanics and computation to enable them to obtain that perspective view of the subject which is essential to real progress. It is very difficult to dogmatise on the question as to what is the best form of education for engineers; many men of broad outlook and wide experience have expressed their views on the subject, and we recommend to the notice of our readers the thought expressed by Professor Swain of Yale University in his recent text-book on "Strength of Materials."

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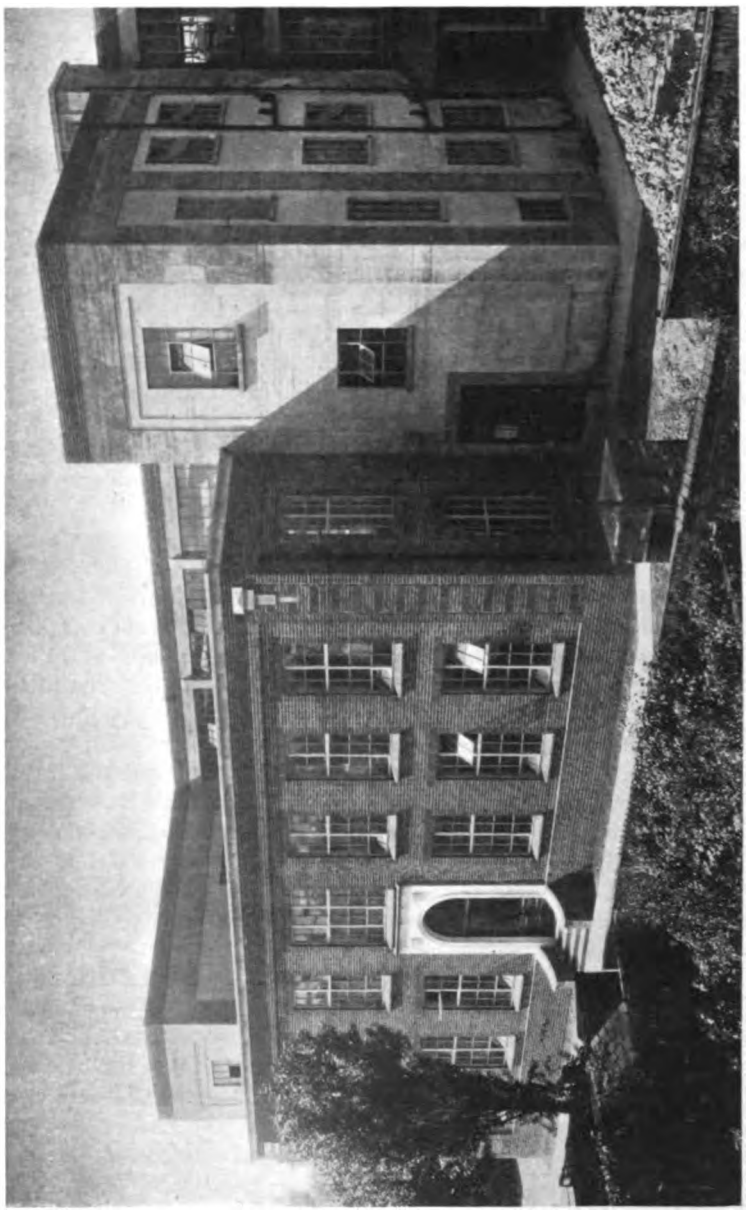

[Messrs. A. F. Scott & Sons, A.A.R.I.B.A., Architects.

New "Flat-Slab" Factory at Norwich: View of Front. (See p. 601.)

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Flat-Slab System at Norwich.

New Reinforced Concrete Building Embodies Latest American Methods.

THE flat-slab method of construction has been in use in America for at least fifteen years, and is almost universally employed for multi-story reinforced concrete buildings; the exceptions being very heavily-loaded floors of warehouses, etc. (where beam and slab construction is found to be cheaper), and very light office and store buildings, where the "dishpan" type of floor is adopted.

The architect of the new factory at Norwich, Mr. E. W. B. Scott, A.R.I.B.A., on visiting the United States and Canada in 1923 for the purpose of studying modern factory buildings, was so impressed with the advantages of flat-slab construction in these buildings that he decided to make use of it, if possible, in this country. Fortunately, the then City Engineer of Norwich (Mr. A. E. Collins, M.Inst.C.E.) interpreted the by-laws in a sufficiently generous manner, and this system was used for the new factory at Norwich for Messrs. Fred Sexton, Ltd.

It is to be hoped that now more than one flat-slab type of building actually stands the regulations which have hitherto prevented the adoption in this country of so valuable a method of building may be relaxed.

The columns have spread, or "mushroom," heads and the slab is thickened and reinforced so as to cantilever out from the columns in every direction and also to act as a beam between the supports. The method is as safe as other types of reinforced concrete when properly designed and executed; but the complicated stresses set up cannot be calculated except empirically. This has been the obstacle to its employment here.

Its more obvious advantages are cheapness resulting from simplicity of shuttering and speed of erection; great rigidity and freedom from vibration; clear ceiling giving free running of pipes, shafting, etc.; and the absence of beam shadows has a marked effect in brightening the interior. These special advantages are even more apparent in wider buildings; the special qualities of the system are at their minimum in buildings only two bays wide.

The factory proper has three stories (with basement under one portion), each floor being 144 ft. by 35 ft. by 12 ft. high. The other portions of the building including staircases and sanitary blocks and offices are of ordinary reinforced concrete construction.

The factory windows are carried close up to ceiling level, the filling below being of red brick. Elsewhere the concrete is exposed, except to the office block. This is faced with red bricks which are built as a permanent outside shuttering to the concrete wall, and are thus semi-constructively used, though carrying no load.

By careful use of these simple means a quiet and dignified appearance with an American flavour has been achieved which is certainly pleasing, and should help to break down the old idea that industrial buildings are necessarily ugly.

The building is arranged for easy extension, one end being temporary and easily removed after the extension has been completed.

The ground floor over the basement is designed to carry 1½ cwt. per sq. ft. live load; the first floor 2 cwt., second floor 1 cwt.; and the roof ½ cwt. Columns are at 18 ft. centres in each direction, and the floor slabs are from 6 in. to 7 in. thick.

The internal columns are hexagonal, as also are the "mushroom" heads, and for economy in centering they are the same size on all floors. Round columns and heads would be preferable, but are expensive when steel forms are not available. The columns are 17 in. across, and the heads are spread to 4 ft. across. Drop panels are 6 ft. 6 in. square and from 11 in. to 12 in. total thickness, the projection under the slab being 5 in. in all cases.

External columns are 15 in. square, and have brackets projecting about 20 in. under the floors.

The slabs are reinforced on the "four-way system," i.e. bands of 13 to 17 ½-in. diameter bars are placed from column to column longitudinally and laterally, and also similar bands placed diagonally, the width of the bands being about one-third of the span, in this case about 6 ft. Half bands are placed around the external edges of the floors, and beams projecting above the floor are provided to support the walls.

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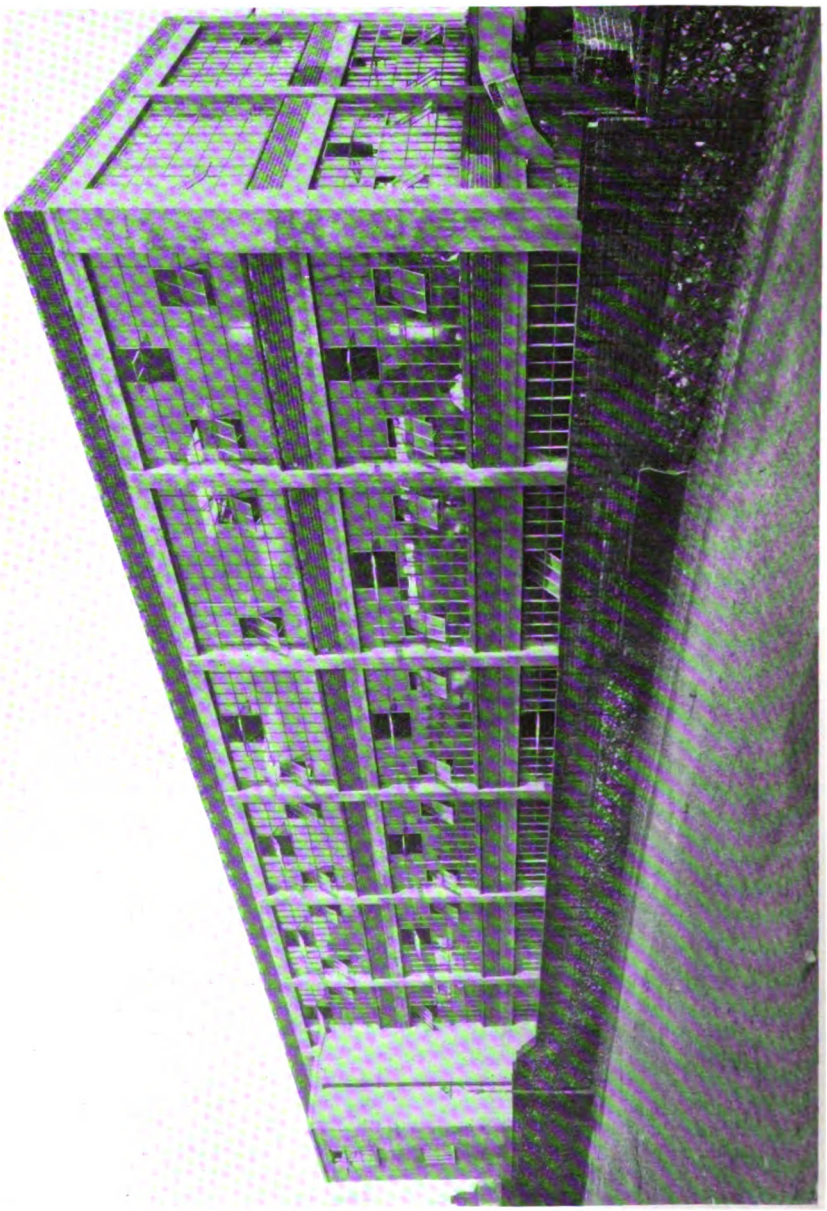

[*Messrs. A. F. Scott & Sons, A.A.R.I.B.A., Architects.*

New "Flat-Slab" Factory at Norwich: Main Factory Block.

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The slab bars are arranged to hang in a catenary between the column heads, and continued about one-third the distance across the adjoining span. Owing to the large number of bars crossing over the columns great care is necessary in placing the steel and concrete at these points.

Some difficulty is met with where large holes have to be formed through the slabs (as for the roof lights) and additional reinforcement is provided to compensate for the lack of continuity due to the holes. In the case of the small hand-lift running through all the floors, the enclosing walls were strengthened and the whole treated as a large hollow column.

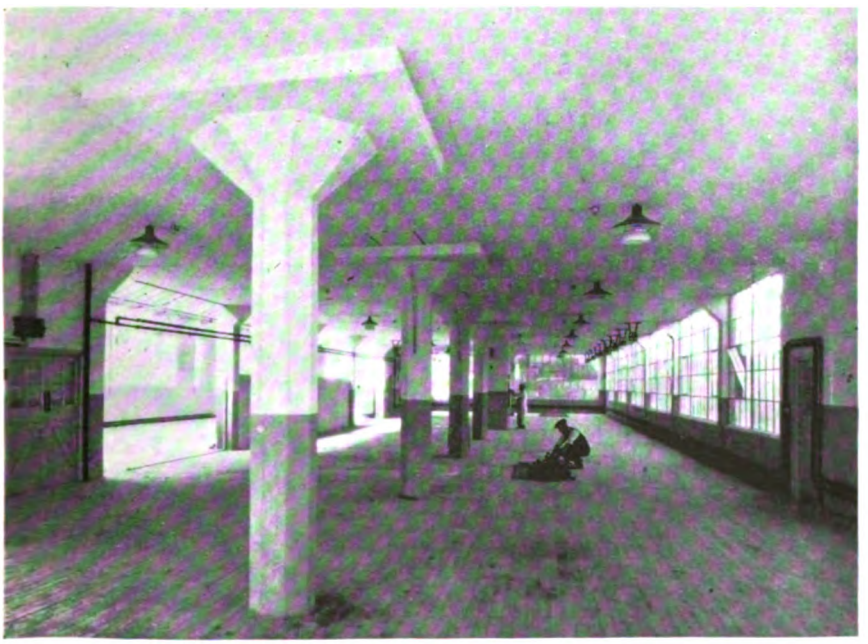
A slightly richer concrete mix than usual was used throughout the work, and in some of the columns and more highly-stressed points 50 per cent. extra cement was added.

Maple floors are provided throughout the factory; roof lights to the top floor; two electric lifts; a cool basement 45 ft. square for storing leather; a system of electrically-accelerated hot-water heating; hot water to wash basins; a new type of warmed hat and coat rails; drinking fountains everywhere; fireproof non-slipping stairways and corridors;—nothing has been omitted which will make for increased efficiency or the health and comfort of employees. The offices are finished throughout in oak.

The electrical equipment deserves separate mention; it comprises the following: (1) Power for motors; (2) Lighting and heating; (3) Electric clocks, time recorders, and workmen's bells; (4) Telephones, and (5) Automatic fire alarms.

Electric power is used throughout the building. Electric motors are fixed to the ceiling so as to give free floor space, driving the shafting which is carried on the usual hangers from special channel fixings, which are let in flush with the ceiling level. The same system is adopted for all conduits carrying wires, etc., so that ceilings and walls are practically free from what is usually an unsightly collection of tubes.

The lighting is arranged on two systems: (*a*) General lighting from high-candle-power gas-filled lamps with special reflectors; and (*b*) Local intensive lighting over special machines, etc. Very great care has been expended on the arti-



New "Flat-Slab" Factory at Norwich: Interior of Typical Floor.

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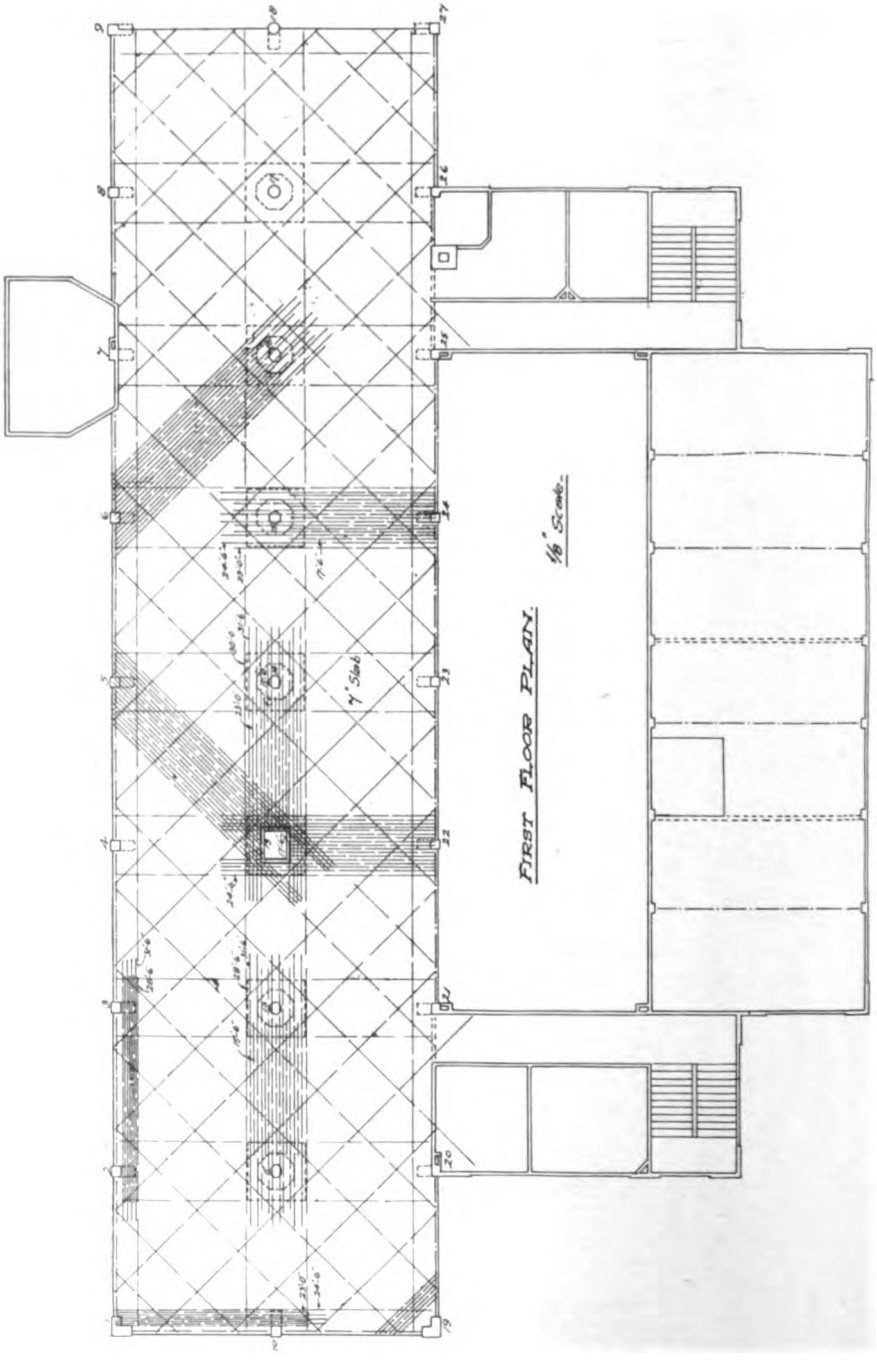

New "Flat-Slab" Factory at Norwich.

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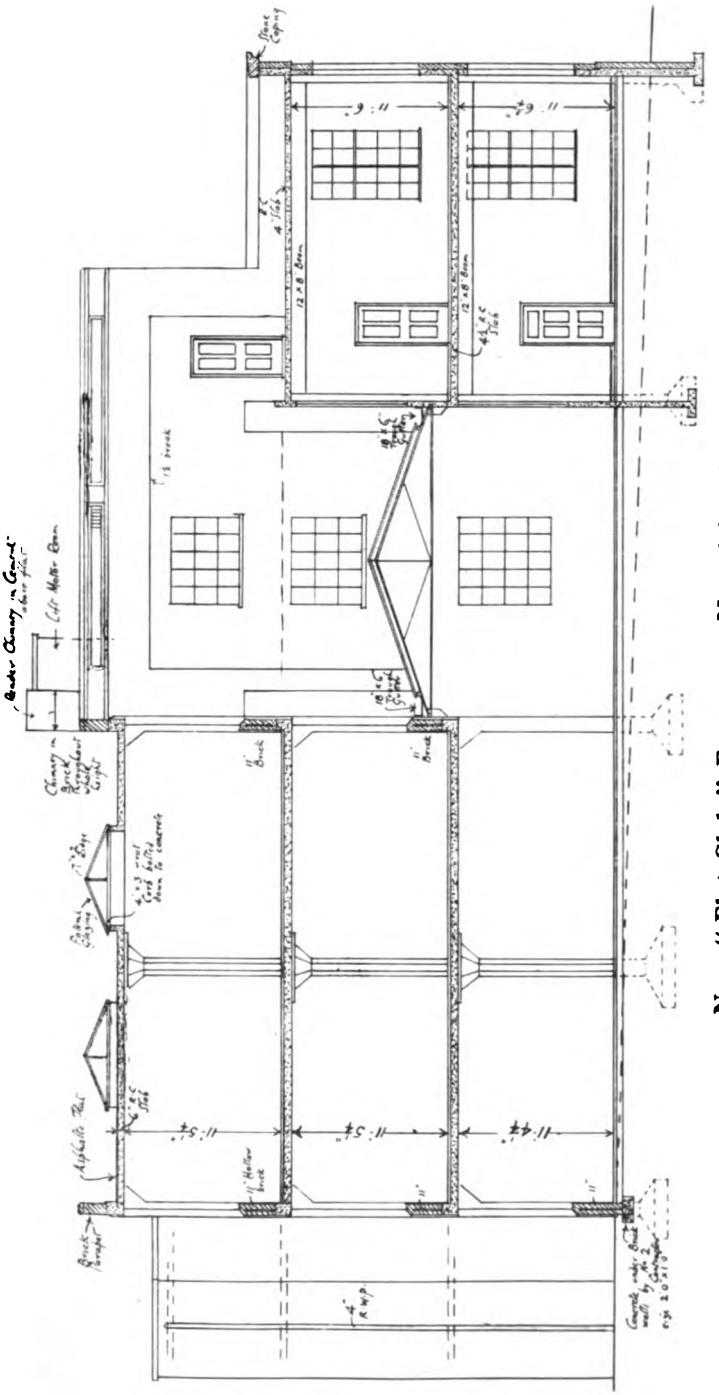

New "Flat-Slab" Factory at Norwich: Section.

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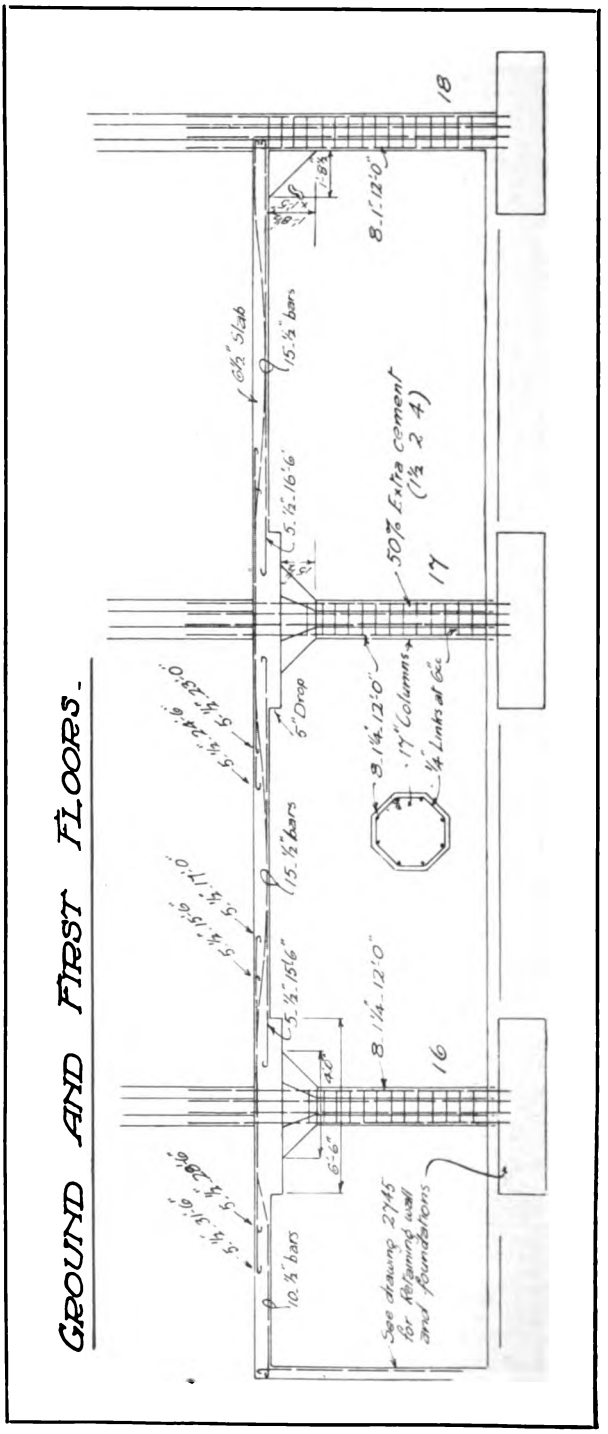

New "Flat-Slab" Factory at Norwich.

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ficial lighting, which differs in important particulars from the ordinary methods.

Electrically-controlled clocks have been fitted throughout, working in conjunction with the works time recorders, and also controlling the works time bells, thus ensuring that the time of starting and leaving off must synchronise with the recorders and the general time clocks.

A complete system of secret-line intercommunication telephones has been fitted, giving direct connection between all departments.

A complete installation of automatic thermostats gives fire protection. The exact position of a fire in the premises is shown on the indicator, an alarm bell outside rings, and at the same moment the alarm is given in the City Fire Brigade station. With this system, which has been adopted in a large number of the most modern and efficiently equipped factories elsewhere, the alarm has been so prompt that the Fire Brigade has actually arrived at the factory where the fire has started before the workers on remote floors have known there has been anything wrong.

All this complication of wiring is buried in the floors, and, as the photograph shows, the ceilings are perfectly clean and free from obstructions.

The architects are Messrs. A. F. Scott & Sons, AA.R.I.B.A., of Norwich, the reinforced concrete engineers Messrs. Lewis Rugg & Co., of London, and the electrical engineer Mr. G. T. Redgment, of Norwich. Mr. A. R. Taylor was the general contractor.

A New Method of Electric Wiring in Concrete Buildings.

WHAT is claimed to be the most up-to-date method of wiring reinforced concrete buildings has been adopted in the new reinforced concrete building now nearing completion in New York for the New York Edison Co. The whole of the wiring was specified on the plans before construction commenced, and tubing, known as "Murray" tubing, laid where ducts were specified. The concrete floors and walls are poured after the tubing is in place and, after the concrete has set, the tubing is withdrawn, leaving a smooth, cylindrical bore running continuously between the desired points. The feeders are carried from the main switchboard up through ducts in the concrete walls to main distributing panels on each floor. Sub-distribution boxes are connected to these panels and the supply groups of outlets, all through ducts in the concrete floors, ceilings and walls. The boxes are formed in the shape of truncated pyramids, while in the moulding of ceiling and floor outlets rubber collars 4 in. in diameter are employed. These boxes and duct connections to the main distributing panels are formed in the rough floor concrete, and when the slab has set and the form beneath removed there remains a finished ceiling with distribution and outlet boxes. The ducts connecting the ceiling outlets with each other and with the distribution boxes are then laid over the rough concrete, terminating in each case with a 4-in. rubber collar. The finer grade of concrete, the finished flooring, is then poured, and over the surface appear the ends of the rubber tubing, sticking up and out of the round openings. Next day these are pulled from their holes and the duct system in that particular part of the building is ready for the electrician.

Lifting a Bridge Span by Tidal Power.

WHEN a reinforced concrete bridge was built at Los Angeles by the Californian State Highway Commission it was borne in mind that it might be necessary to open the centre span, and it was designed accordingly. It was recently desired to pass a dredge under the bridge, and for this purpose the centre span, 46 ft. long and weighing 168 tons, had to be temporarily removed. The lifting was done by tidal power. A barge, with 12 in. by 12 in. timbers bolted to the deck to distribute the load, was placed in position under the span at low tide, and as the tide raised the barge so the span was lifted. The barge measured 30 ft. by 104 ft. by 9 ft. deep, with a draught of 2 ft. 6 in., and was submerged 23 in. before lifting of the span commenced. The barge with its load was floated out and made fast alongside the bridge, while the tug passed through, and then towed back and the span replaced as the tide receded.

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Concreting in Cold Weather.

THE following suggestions for protecting concrete poured during cold weather are put forward by the American Portland Cement Association.

Always be prepared to enclose the work and supply heat whenever work that may run into cold weather is begun.

Aggregates are heated to best advantage by 1½-in. steam pipes laid as a grill under the piles or by a 6-ft. length of perforated steam pipe inserted into the pile.

Water is heated by running a 1½-in. steam pipe into the water barrel.

A 50-h.p. boiler, carrying 50 to 60 lbs. of steam, is required for a large job, but an 18 to 25-h.p. boiler is sufficient for jobs of not more than 5,000 sq. ft. of floor area.

One fire bucket burning coke is usually sufficient for 300 sq. ft. of floor area, and a heat hole should be provided for each fire bucket.

Columns should be concreted at the same time as the floor, and column tamping continued for an hour or more after the floor is concreted.

Bases of exterior columns are most difficult to protect, and in extremely cold weather a fire bucket should be placed on each of two sides of exterior columns.

For unusual speed or cold weather, or when monolithic walls are carried up, the lower floor should be curtained and fire buckets placed at exterior columns.

Apply fuel to fire buckets frequently and in small quantities, to minimise smoke and provide uniform temperature.

Water barrels should be provided to extinguish fires, and care taken to keep canvas from blowing against fire buckets.

A detailed temperature record should be kept showing date, hour, outside temperature, temperature at bottoms of columns, underside of slab, under the canvas over the slab, temperature of concrete as deposited and, especially, temperature at bases of exterior columns on windward side of building.

Aim at a minimum temperature of 60 degrees F. within the enclosure for five days. During protracted cold weather keep the tarpaulins up and maintain a temperature of not less than 40 degrees F. for ten days additional.

Do not remove permanent shores until the strength has reached three-fourths the design-strength of the concrete.

Do not depend on anti-freezing compounds instead of protection and heat. Accelerators do not furnish heat to start the hardening process, but once started, accelerators will cause the concrete to gain strength faster if it is kept warm. No accelerators or anti-freezing compounds should be used that will decrease the final strength of the concrete, promote rusting of reinforcement or produce efflorescence.

Professional Registration.

SOME time ago a movement for the registration of architects was set on foot in the Straits Settlements with the object of precluding anyone except members of the Royal Institute of British Architects from carrying out architectural work in that country. With the objects of the movement few will disagree, for engineering has suffered as much as architecture from the work of unqualified practitioners. But the local by-laws in the Straits Settlements include civil engineering, structural engineering, and surveying under the heading of "architectural work," so that should this Architects' Registration Bill have become law no one but an architect would have been able to practise engineering. We are glad to see, therefore, that the bodies representing these professions have vigorously taken the matter up with the Colonial Office, with good results, and that a clause is to be embodied in the Bill to the effect that it shall not be construed as debarring any person who is a Chartered Civil Engineer, a Bachelor of Engineering, a Bachelor of Science of any British or British Colonial University, a member of the Institution of Structural Engineers, the Royal Sanitary Institute, the Institution of Mechanical Engineers, the Institution of Electrical Engineers, the Surveyors' Institution, the Institution of Mining and Metallurgy from exercising his functions as a civil, structural, sanitary, mechanical, electrical, or mining engineer, or as a surveyor.

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A New Revision of the British Standard Specification for Portland Cement.*

THE latest edition of the British Standard Specification for Portland cement embodies more alterations and greater advances than any of the four preceding editions, as will be seen from the following summarised comparison of the principal requirements of the new Specification and its immediate predecessor:

		1925 Edition.	1920 Edition.
Fineness—			
Maximum Residue—			
180 × 180 sieve		10 per cent.	14 per cent.
76 × 76 ,, ...		1 ,,	1 ,,
Chemical Composition—			
$\frac{CaO}{SiO_2 + Al_2O_3}$ . . .	Limits	2·0 to 2·9	2·0 to 2·85
Insoluble residue, maximum . .		1·5 per cent.	1·5 per cent.
Magnesia ,, . .		4 ,,	3 ,,
Sulphur (as SO_3) ,, . .		2·75 ,,	2·75 ,,
Loss on ignition ,, . .		3 ,,	3 ,,
Tensile Strength—			
7 days neat, minimum . . .		600 lbs. per sq. in.	450 lbs. per sq. in.
28 ,, ,, ...		abandoned	7 days result. $+ \frac{40,000}{7 \text{ days result}}$.
7 days—3 sand, 1 cement, minimum . .		325 lbs. per sq. in.	200 lbs. per sq. in.
28 ,, ,, ,, . .		7 days result. $+ \frac{10,000}{7 \text{ days result}}$.	7 days result. $+ \frac{10,000}{7 \text{ days result}}$.
Setting Time—			
Initial set, minimum . . .		30 minutes.	20 minutes.
Final set, maximum		10 hours.	10 hours.
Soundness—			
Le Chatelier test, maximum . .		10 m.m.	10 m.m.

Dealing with these items in order it will be seen that a greater degree of fineness is required. This is not of much importance in itself, and the old maximum limit of 14 per cent. on the 180 sieve has probably been abandoned because present-day cements are almost invariably below the new limit of 10 per cent.

The upper limit in the molecular ratio of lime to silica and alumina is raised from 2·85 to 2·90. This is a useful alteration because it gives the manufacturer of rapid-hardening cement more scope to produce a still better product. In the present state of knowledge there is no reason why the lime ratio limit should not reach the theoretical figure of 3·0, because there is evidence that sound cements can be made up to this limit, and to insist on any lower figure tends to retard the further development of cement quality.

* British Standard Specification for Portland Cement (Revised 1925). London: Crosby Lockwood & Son. Price 1s. net.

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The increase in the permissible proportion of magnesia from 3 to 4 per cent. widens the scope of raw materials that can be used for cement manufacture, and is therefore beneficial. There is abundant evidence from American and other experience that 4 per cent. of magnesia in cement is in no way deleterious.

It is in strength requirements that the most revolutionary changes have been made. The minimum for the 7-day neat test has been raised from 450 to 600 lbs. per sq. in., and the 28-day neat test has been abandoned. The insistence on an increase in the neat test between 7 to 28 days has more than once been assailed in these pages as reactionary, because cement manufacturers were hindered in their efforts to produce rapid-hardening cement owing to the necessity for showing a growth in the neat strength between the ages of 7 and 28 days. The neat test for cement is only of laboratory interest, and it seems probable that the deletion of the 28-day test is the first step towards dropping the neat test altogether, as has been done extensively in other countries.

The minimum for the 7-day sand test has been raised from 200 lbs. to 325 lbs., and this marked advance places the British Standard Specification on a pedestal high above all its national contemporaries for severity of requirements. Coupled with the safeguards for soundness, Portland cement of this quality can only be produced by careful and scientific manufacture; and the cement user is protected against the imposition of inferior cements which were able to creep into the fold when the previous limits were in force and claim equality in passing the same Specification as brands having double their strength. Although there are brands of British cement on the market which invariably show over 400 lbs. sand test at 7 days, and whose manufacturers might have welcomed a minimum of 400 lbs., yet there are others which have not reached this standard, and for the time being 325 lbs. can be accepted as a standard that will eliminate some of the inferior cements which just managed to comply with the old standard.

So far as setting-time is concerned, the minimum initial set is increased from 20 to 30 minutes. This is no hardship to the manufacturers, and gives the user greater scope for permitting longer delays and hindrances to the mixing and laying of concrete without endangering "killing" the cement by interference with the initial set.

It is to be regretted that the Standards Committee has not been able to discover a better means than now exists for defining the amount of water to be used for setting-time and tensile tests. In a foreword to the Specification it is stated that "extensive investigations" have been carried out, but the problem remains unsolved and this fruitful cause of divergency in results obtained by individual testers still remains. The setting-time and tensile tests of cement depend upon the consistency of the paste used for moulding the test pieces, and a method of defining a standard consistency is badly needed. The setting-time test itself is one that demands further investigation because the "initial set" as now recorded merely indicates a degree of stiffening which is not the same as the commencement of setting, while the "final set" is recorded long before the cement is hard and in reality only coincides with a greater degree of stiffening.

Another important innovation is the clause that cement intended for use in hot climates must have the specified setting times when tested at a temperature

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up to 95 deg. F. This is a common-sense measure of precaution for cement-users in the tropics.

The proportion of water to be used for the sand tests is in future to bear a definite relation to the proportion used for the neat tests.

No compression tests are included in the Specification, although resistance to compression is the most rational test that could be devised for cement. There is probably some justification for this omission in view of the cumbersome and costly apparatus required for compression tests, but in future revisions the inclusion of standards for compression tests might well be considered leaving the adoption of the tests to be a matter for arrangement between cement buyer and seller.

The Specification is published in the same form as heretofore, but a useful summary of the tests is now inserted.

A Concrete Powder House.

[This powder house of a United States mining company is of interest in that its concrete construction has been so carried out that if an explosion should occur in it the structure would merely disintegrate instead of flying through the air in every direction. No large aggregates were used in the construction so as to reduce to a minimum the damage done in case of an explosion. The least possible amount of reinforcement was required due to the catenary shape of the building.]

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Concrete Piles 110 ft. Long.

In the construction of a large new shipping pier at Manilla use is being made of reinforced concrete piles 110 ft. in length and 24 in. by 24 in. in section. All the piles are poured from an aerial cableway, the pile field accommodating over 1,500 piles of various lengths. A 1 : ½ : 3 mixture was used, test cubes of which gave strengths of from 3,600 lbs. to 5,300 lbs. per sq. in. at from 28 to 40 days.

The piles are reinforced with eight 1-in. deformed bars with the centre of the steel 2½ in. from the surface. Hoops of ¼-in. rods were used with a spacing of 1½ in. at the head of the pile for a distance of 3 ft., then 3, 6 and 9-in. centerings over a distance of 1, 2 and 3 ft. Twelve-inch spacing was satisfactory for the rest of the length of the pile except near the point of the pile, where the hoops were spaced on 3-in. centres for a distance of 3 ft. All bars were allowed to project 3 ft. above the head of the pile for the purpose of rigidly connecting the deck girders to the pile, while at the points the bars were bent inward to give greater strength. A 1½-in. rod, bent in the shape of a hairpin, was set in the head of each pile to receive the shackle of the pile line used in suspending the pile in the leads of the driver. Cambered piles were cast for use as batter or brace piles, and were reinforced as the straight piles except that the reinforcement was considerably heavier in the plane of camber. The camber in the longest batter pile amounted to approximately 5½ inches.

The piles were transported to the pile-barge by a 50-ton locomotive, and driven by what is claimed to be the most powerful floating pile driver ever built mounted on a wooden hull 110 ft. long and 55 ft. wide. The longest piles weighed 64,000 lbs. The hammer used for driving is 17 ft. high, 36 in. across the face, 26 in. deep, and weighs 30,000 lbs. The cylinder has a bore of 14 in. and a 36-in. stroke and operates a 4,000-lb. ram. The piston is double-acting, and with an effective steam pressure of 80 lb. the hammer develops approximately 4,000,000 ft. lb. of work per minute, striking at the rate of 85 blows. The follower is of cast steel and is designed to fit over the hairpin and reinforcing rods which project above the head of the pile. A spiked wood cushion block of native hardwood, which is very tough and fibrous and which is banded with ¼-in. flat steel 4 in. wide and 18 ft. long, wrapped three times around and riveted with countersunk rivets closely spaced, fits on top of the follower and receives the direct blows of the hammer. Cushion blocks last on the average for the driving of six piles. Where the driving has been very hard some blocks have given out after driving a single pile. A maximum of eight piles per 10-hour day has been driven.

As the depth of the hard material varied considerably, a test load of 200 tons was applied to a few piles whenever a considerable amount of variation in penetration occurred. Under the maximum loading these piles are driven to support, and an apparent settlement of 11 mm. (about $\frac{21}{64}$ in.) occurred, part of which was deformation. Under an additional loading of 100 tons a further settlement of 11 mm. was recorded. Upon removal of all load, after the lapse of 15 days, a recovery of 8½ mm. (about $\frac{21}{64}$ in.) was noted. Had the pile not been loaded beyond 100 tons the net settlement realised would probably have amounted to 3½ mm., or approximately $\frac{5}{64}$ inch.

The pier is being built by the Bureau of Public Works of the Philippine Islands, under the supervision of Mr. E. C. Earle.

A Tall Concrete Building.

The tallest concrete building in the British Empire has been completed at Toronto. It is the Northern Ontario Building, which has 16 stories and covers an area of 8,000 square feet. All the walls and floors are of reinforced concrete, and the pouring was carried out at such a high speed that on the average a story was built each week.

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Problems of Cement Manufacture.

THE fact that hardly a week passes without the publication of one or more patents dealing with cement manufacture is evidence that there are considered to be numerous possibilities of improvement in the cement industry, and although it is true that patented processes, or even patented devices, used in the manufacture are rarely found in cement works, the problems of cement manufacture are apparently so obvious as to be continually exercising the minds of inventors.

The purposes of the patents may be classified under three headings :—

(1) To reduce the cost of manufacture of cement as now made ;

(2) To improve the efficiency of the existing processes of cement manufacture ;

(3) To produce new cements.

The process of Portland cement manufacture consists of (*a*) mixing two or more raw materials to a high degree of intimacy, (*b*) heating the mixture to about 2,700 deg. F., (*c*) grinding the semi-fused product to a fine powder.

The process of mixing raw materials provides ample opportunity for the inventor. With the wet process now generally adopted it is necessary to add water to bring the mixture to a liquid condition before the desired degree of consistency of composition can be obtained, but as this added water must be evaporated in the heating process (forming the next stage of manufacture) at the expense of fuel, it is evident that this feature of the wet mixing process is a direct charge on the cost of manufacture.

An alternative is the dry process of manufacture whereby the raw materials are dried, ground together to a fine powder, and then mixed as well as possible. It is in the mixing of the ground dry raw materials that the dry process has its weakness, because it is practically impossible to mix powders in the huge quantities required for cement manufacture to the degree of intimacy that is desirable for the production of cement up to the present-day standard of quality.

With materials that will flow, such as the slurry prepared in the wet process, it is possible to obtain day by day a mixture of which the principal constituent—carbonate of lime—does not vary by more than one-tenth per cent. from the standard aimed at. But with the dry process it is exceptional if the divergence from standard composition does not exceed one-quarter per cent., and even this consistency of composition is only secured by heavy capital outlay in mixing and storage plant. Hence cement manufacturers, in default of better methods, have to choose between the wet process of raw material mixing with its high fuel consumption for calcination, and the dry process with its lower standard of quality.

The ideal mixing process would be one in which the raw materials, usually chalk or limestone and clay or shale, would be taken as they are excavated from the earth and ground together without addition of water and without drying until a homogeneous mixture was obtained. When the physical characteristics of damp chalk and plastic clay are considered this may seem to be an impossible task for the inventor, but remembering that coal containing up to 15 per cent. of water can be ground to a fineness fit for boiler firing it is a problem that cannot be summarily dismissed as insoluble.

Failing this, however, a machine would be welcomed that would take powders such as are produced in the dry process and mix them to the almost perfect degree

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of consistency required for the cement industry. Such a machine would need to deal with at least 10 tons of material per hour, and must neither be expensive to maintain nor absorb much power.

Another partial amelioration of present conditions would be a cheap mechanical means of reducing the proportion of water in the raw material slurry produced by the wet process. Such slurry contains from 35 to 45 per cent. of water, and, as previously stated, this water has to be driven off in the calcining kiln at the cost of fuel. If it were possible to remove the greater part of this water by a mechanical method, such as would not involve much power consumption or any manual labour, there should be a possibility of economy in the calcining stage of the manufacture.

The calcining stage of cement manufacture has already been the subject of numerous patents, but perfection in this department is still far ahead. The object of the calcination is to cause silica and alumina to combine with lime to form certain chemical compounds, and the process requires a temperature of at least 2,500 deg. F.

The combination is taken to be complete when the mixture commences to fuse and a coagulated material known as " clinker " is formed. There is, however, no known reason why the calcination should not be carried to the stage of fusion, and it is probable that this would yield a higher quality of cement while it would have the advantage of easing the requirements of the raw material mixing process, because it would not be necessary to grind the raw materials so finely nor mix them so evenly if the mixture were subsequently liquefied in the kiln.

The temperature required for the fusion of cement clinker is, however, very high (probably over 3,000 deg. F.), and the practical difficulties of maintaining this temperature at a reasonable cost have not yet been overcome. The problem is one that has been under consideration for many years, and there are some who consider that the cement of the future will be obtained by complete fusion.

Returning to the calcination methods now employed, the inquiring mind is impressed by the fact that in the rotary kiln—the kiln principally used—the fuel consumption is about four times as much as is theoretically needed. The distribution of the surplus heat is well known ; some of it is lost in radiation from the kiln, more is absorbed in evaporation of water from the raw material mixture, and much passes up the kiln chimney. Many have been the inventions either to prevent the loss of heat from the kilns in the directions described or to utilise the heat, and definite improvements have taken place during the last ten years, but with a rotary kiln efficiency of only 25 per cent. there is obvious room for further economies.

Some of the inventions for fuel economy in the rotary kiln, while effective in the direction aimed at, have had to be rejected because of other difficulties they have introduced. For example, it is not impossible to insulate a rotary kiln shell to prevent loss of heat by radiation, but when this is done it is found that the lining bricks in the kiln have a short life because the bricks commonly employed owe their endurance to the fact that they are cooled by radiation. Or again, it is possible to modify a rotary kiln so that the temperature of the chimney gases is reduced and less heat is wasted, but with present knowledge such modifications can only have a restricted scope, otherwise the producing capacity of the kiln is impaired.

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There is more scope for economy in the rotary kiln process of burning than in any other stage of cement manufacture, and doubtless the flood of inventions dealing with the rotary kiln will continue; but the possibility of the rotary kiln being superseded by a kiln producing a fused product must not be overlooked.

The subject of grinding, which is the final stage of cement manufacture, has also received the attention of inventors, and mills of many types have run the gauntlet of cement works' practice. The weeding-out process has been severe and the tube mill will apparently be the chief survivor, or possibly in a few years' time the only survivor, the reason being that in a cement works simplicity is a great asset. Unfortunately there are no published methods of measuring the efficiency of a tube mill—that is, of comparing the power absorbed in grinding a ton of material with the power theoretically required. There is reason to believe that the energy needed to break up a ton of clinker to the fineness of cement is a much lower amount than that actually expended in a tube mill, and if this is so the tube mill is capable of improvement or of being superseded by a mill which more usefully employs the energy put into it without being complicated.

There now remains to consider the field of discovery for the invention of new cements. Many of the attempts in this direction have been concerned with the employment of blast-furnace slag, but with no particular success. The discovery of aluminous cement is a conspicuous example of a good invention likely to be of permanent value.

The incentive for the discovery of new cements lies in the desire to produce something cheaper than Portland cement and, if possible, something of better quality. Assuming that lime will always be the basis of any constructional cement, economy would be secured if cement could be produced with, say, 40 per cent. of lime instead of the 62 per cent. now essential in Portland cement. Such a low-limed cement would require less fuel for calcination, and as it would probably fuse at a low temperature the raw material grinding and mixing would not require to be so thorough as is now necessary. But up to the present no appreciable cementitious properties are associated with such low-limed compounds.

Another desirable improvement would be a cement of light weight. A cubic yard of ordinary cement weighs about 1 ton, and in the usual 1 : 2 : 4 concrete will make about 4½ cubic yards of concrete. If, however, it were possible to produce a cement weighing half a ton per cubic yard (41 lbs. per cubic foot) with the same strength as modern Portland cement, then a ton of cement would make 9 yards of concrete and considerable economy would result.

While the possibilities of development in connection with the manufacture, characteristics and properties of Portland cement can be the subject of interesting speculation, and rightly receive the attention of many inventors, it is not intended to leave the impression that cement manufacturers have failed to progress either in manufacture or in quality. The rapid-hardening cements of to-day are a fine illustration of the progress in quality, and the scientific precision with which rotary cement kilns are managed is an example of the high status that has been achieved with existing methods.

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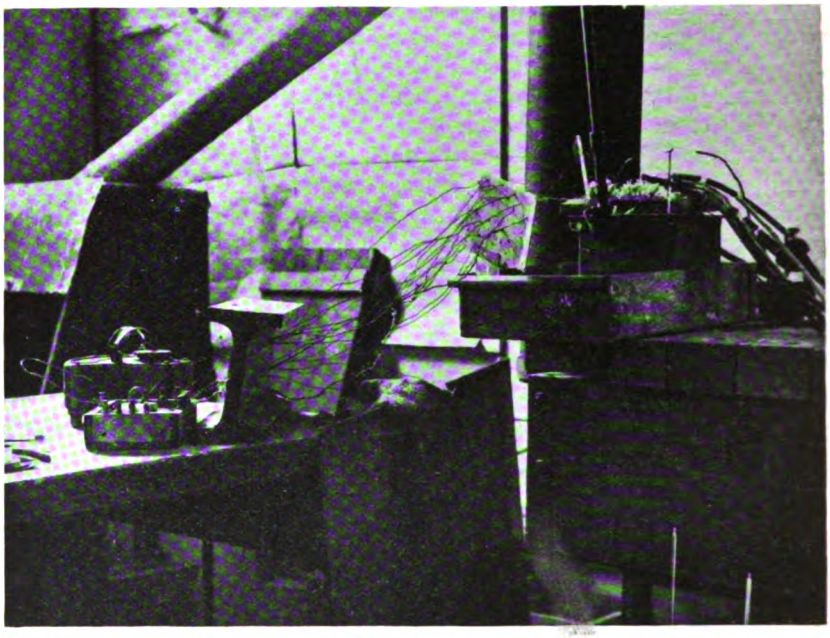

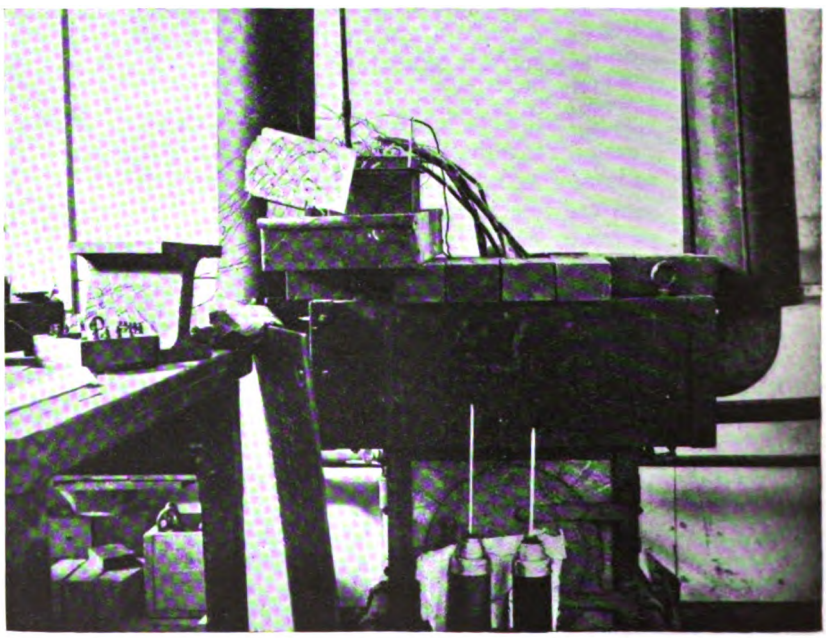

Fire Tests on Concrete Slabs: Method of Temperature Recording.
(*See p.* 617.)

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Fire Tests on Concrete Slabs.

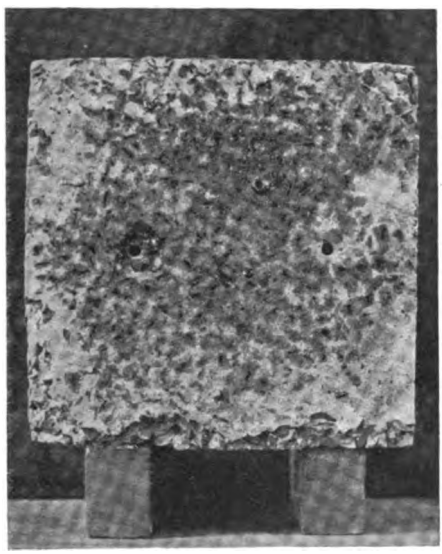
By Robert C. Panton, M.A., M.Sc., A.M.Inst.C.E.

(Lecturer in Civil Engineering at the University of Birmingham.)

THE author has, during the last four years, carried out extensive tests at the University of Birmingham upon the behaviour of concrete slabs subjected to high temperatures, and this short report endeavours to furnish in as few words as possible the essential information obtained from these tests.

A special air-blast furnace with D.C. motor blower, made by Messrs. John Wright & Co., was kindly lent by the Department of Scientific and Industrial Research to enable these tests to be carried out. In nearly all the slabs tested, the ratio of cement, sand and aggregate was 1 : 2 : 4 by volume, the aggregate being Rowley Rag ¾ in., while the sand employed was either Leighton Buzzard or Drainage Board twice washed, the cement being in all cases "Ferrocrete" supplied by the Cement Marketing Company.

As these ingredients make a fairly average concrete, it is thought that the results obtained may be applicable to most ordinary concretes. Iron and iron constantan thermocouples in conjunction with a direct-reading pyrometer were employed for reading the temperatures at various distances inside the slabs. These thermocouples were satisfactory, if used with care, up to temperatures of about 1,000 deg. C. Very considerable practice and skill was required in obtaining readings beyond 1,000 deg. C., and for further research couples of materials such as nickel and bright ray are advised.



Slab after Test.

Rowley Rag has begun to melt.

The results may be summarised as follows :—

Slabs 18 in. × 18 in. × 4 in. thick.
18 in. × 18 in. × 6 in. thick.
18 in. × 18 in. × 8 in. thick.

Inner face of the slab exposed to a uniform temperature. Outer face of the slab exposed to ordinary air temperature.

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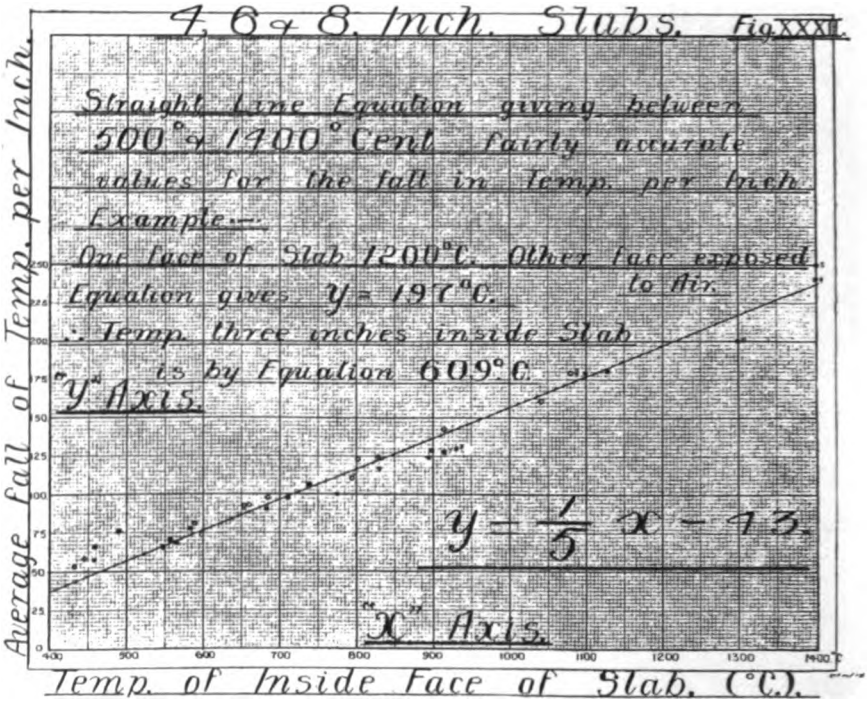

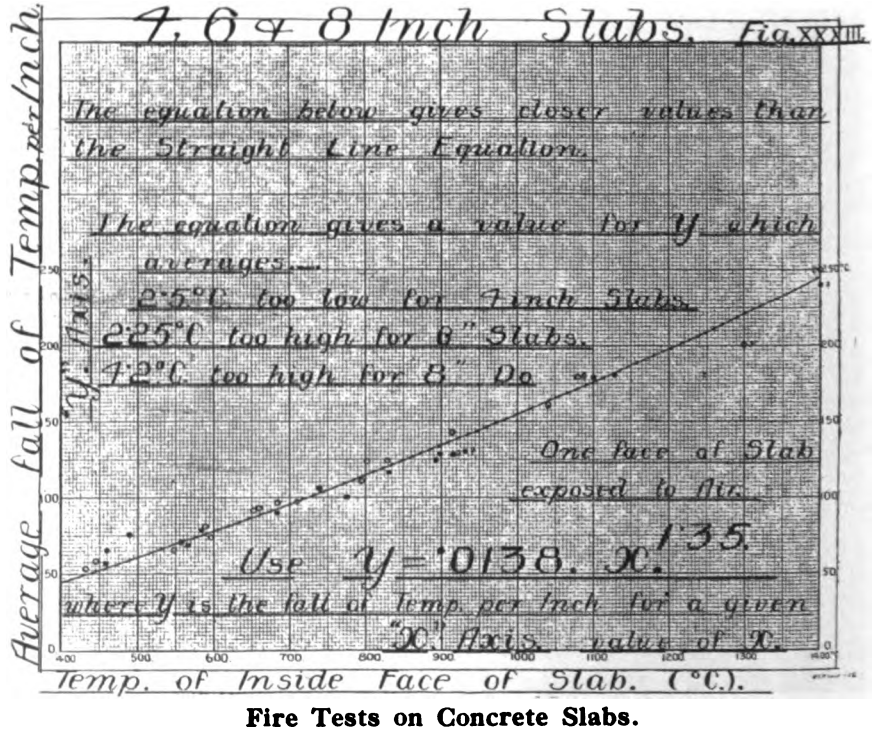

Fire Tests on Concrete Slabs.

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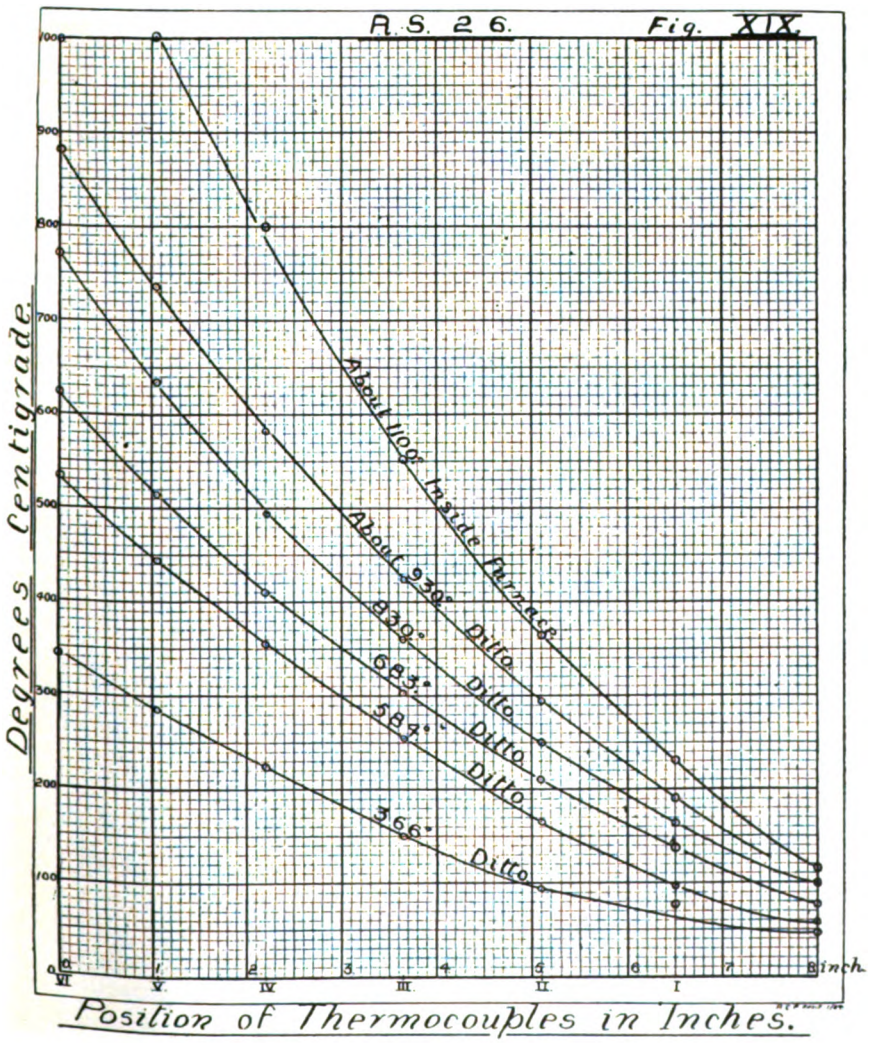
(1) For any definite furnace temperature, when the flow of heat through the slab has become steady, the fall in temperature per inch increases the nearer the inner face of the slab is approached. Expressing this more generally, the thermal conductivity diminishes at higher temperatures.

(2) If the furnace temperature is increased and the flow of heat through the slab is again allowed to become steady, the fall in temperature per inch for each corresponding inch through the slab is increased.

(3) For furnace temperatures between 400 deg. Centigrade and 1,400 deg. Centigrade, the average fall of temperature per inch for the first three inches or so of slabs of the dimensions and under the conditions mentioned above, may be calculated from the straight line equation

$$Y = 1/5 . X - 43$$

where Y is the fall of temperature per inch in degrees Centigrade and X is the temperature of the inner face of the slab, also in degrees Centigrade.



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(4) A more accurate equation is

$$Y = \cdot 0138 \,.\, X^{1\cdot 35}$$

Y and X having the same meaning as in § 3.

(5) A still more accurate estimate for Y may be made by correcting the equation in § 4 by the following amounts :—

add 2·5 deg. C. to Y in § 4 for 4 in. slabs

subtract 2·25 deg. C. from Y in § 4 for 6 in. slabs

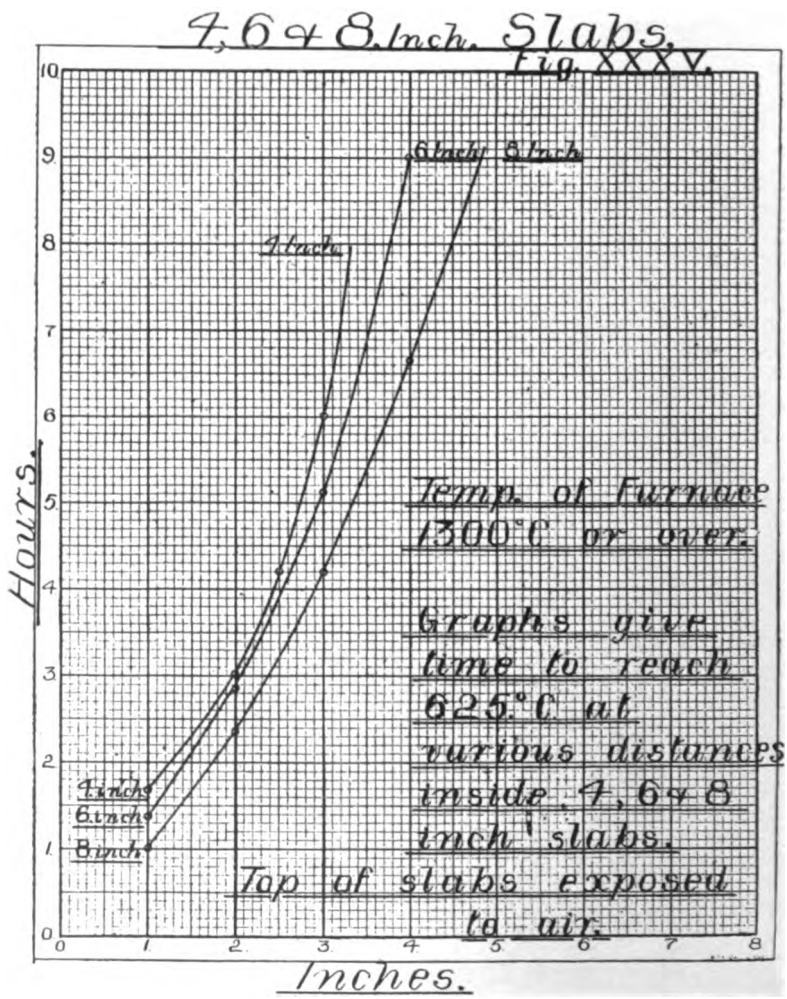
subtract 4·2 deg. C. from Y in § 4 for 8 in. slabs.

(6) If a slab, the outer surface being left exposed to ordinary air temperature, has its inner surface rapidly raised to a temperature of 1,300 deg. Centigrade or over, but not exceeding 1,400 deg. C., and if the inner surface is exposed to this high temperature for a considerable time, the temperature of 625 deg. C. is reached at a distance one inch from the inner surface in :

H.	min.		in.
1	40	in the case of a	4 slab.
1	22	,, ,, ,, ,, ,,	6 ,,
1	0	,, ,, ,, ,, ,,	8 ,,

Similarly, 625 deg. C. is reached two inches from the inner surface in :

H.	min.		in.
3	2	in the case of a	4 slab.
2	51	,, ,, ,, ,, ,,	6 ,,
2	21	,, ,, ,, ,, ,,	8 ,,



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625 deg. C. is reached three inches from the inner surface in :

H.	min.		in.
6	0	in the case of a	4 slab.
5	8	,, ,, ,, ,, ,,	6 ,,
4	12	,, ,, ,, ,, ,,	8 ,,

625 deg. C. is reached four inches from the inner surface :

Never in the case of a 4 in. slab.

H.	min.		in.
in 9	0	in the case of a	6 slab.
6	39	,, ,, ,, ,, ,,	8 ,,

625 deg. C. is reached five inches from the inner surface :

probably never in the case of a 6 in. slab,

in about 10 hours 0 minutes in the case of an 8 in. slab.

(7) The age of the concrete does not appear to affect results to any appreciable extent.

The following examples show how the information in paragraphs 3, 4, 5 and 6 may be usefully employed.

Example 1.—If the inner surface of a wall made with an ordinary concrete is exposed to a temperature of 1,000 deg. C., the other face being exposed to the air, what is the temperature 3 inches in from the inner face after the wall has become thoroughly heated ?

Paragraph 3 gives the drop in 3 inches as 471 deg. C. ; therefore the required temperature is 529 deg. C.

Paragraph 4 estimates more accurately the temperature as 535 deg. C.

Paragraph 5 estimates still more accurately the temperature as

527·5 deg. C. for a 4 in. wall ;
541·75 deg. C. for a 6 in. wall ;
547·6 deg. C. for an 8 in. wall.

Example 2.—A fire breaks out on the ground floor of a reinforced concrete building. This fire attacks the ceiling, which may be considered as the inner face of the floor of the room above. Assuming that the temperature of the fire is 1,400 deg. C., and that the other face of the floor is exposed to air temperature, how soon will the steel reinforcement of the floor, which is 1½ inches from the face exposed to the fire, reach a temperature of 625 deg. C. ?

The graphs in *Fig.* 35 give the time as

H.	min.		in.
2	18	for a	4 floor.
2	7	,, ,,	6 ,,
1	40	,, ,,	8 ,,

A few photographs and graphs, chosen from about 40 supplied, are reproduced for the purpose of making the text of this short report as clear as possible.

Reinforced Concrete Wharf for Zanzibar.

An important step in the construction at Zanzibar of a reinforced-concrete wharf 800 ft. in length has been reached with the driving of the last of the reinforced-concrete piles on which the main structure is to be erected. Work was begun on the wharf five years ago, and it is estimated that two more years will be required to complete it.

The wharf, which has been designed to accommodate ocean-going vessels at low-water ordinary spring tides, will greatly increase the facilities for large vessels operating on the East African coast. The piles and beams were pre-cast, and native labour was trained for the work. Four hundred piles have been driven, ranging from 49 to 72 ft. long and weighing from 5¾ to 9 tons. In some cases where a soft patch was encountered the piles had to be lengthened *in situ* after being driven, and when re-driven reached a maximum length of 84 ft. A breakwater is under construction to protect the harbour from the north-east monsoon.

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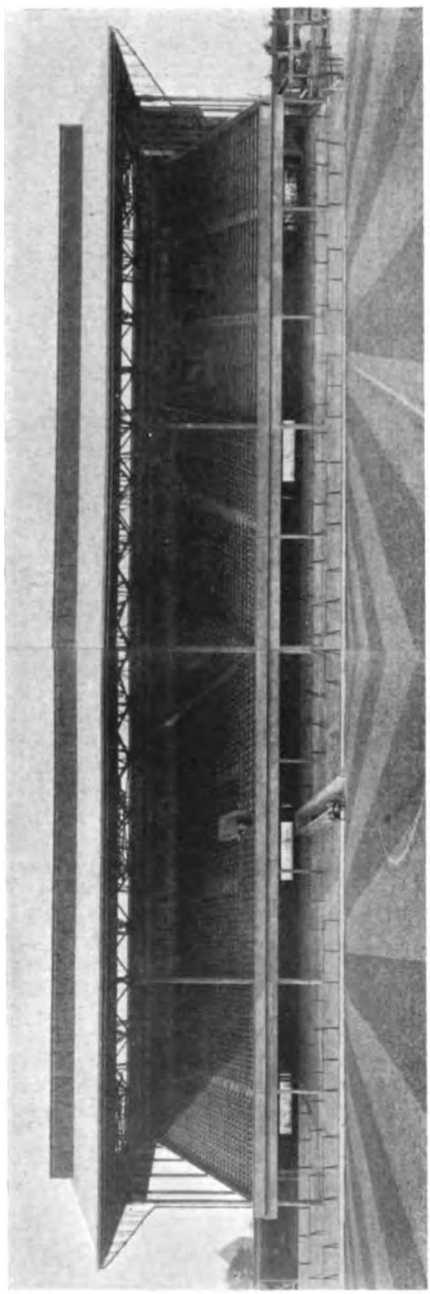

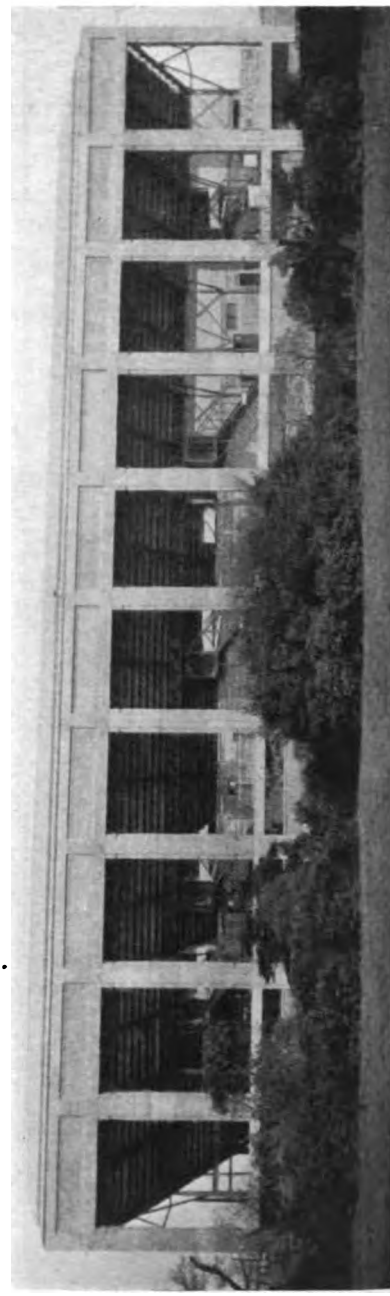

New Grand Stand of the West Ham United Football Club: Front and Rear Views. [*Sir Owen Williams, K.B.E., B.Sc., Engineer.* (*See p.* 623.)

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New Grand Stand for the West Ham United Football Club.

THE growing popularity of the West Ham United Football Club compelled the Directors to consider the provision of improved accommodation for the spectators. The removal from the existing site was considered inadvisable, and only a comparatively small extension of the site could be negotiated with the adjoining owners. Working within this limitation of space it was decided to construct a stand which would give increased seating accommodation and at the same time increase standing accommodation under cover.

To achieve this it was necessary to adopt the double deck construction, i.e. the seating accommodation is actually over the standing accommodation. In further pursuit of the idea of maximum protection it was decided that the roof of the stand should project to within 1 ft. of the "touch line." A further factor in the design was the existing stand, which had to be maintained during last season whilst the new stand was under construction.

The stand is 350 ft. long, 100 ft. deep, and 80 ft. high, and accommodates 10,000 people on its standing terrace, and 5,000 on the seating terrace, all under cover. Under the terraces are the players' quarters with a covered running-track for the training of the players during inclement weather. Refreshment buffet, stores, heating chamber, drying room, etc., are also on the same level. At a higher level under the seating terrace are the Directors', officials', and press rooms, with refreshment buffet and sanitary accommodation.

The entrance to the stand is by a 50 ft. private road leading to one end of the stand, where the main turnstiles are situated. The exit staircases total 16 ft. 9 in. in width for the seating and 29 ft. for the standing accommodation, and actual observation shows that there is no difficulty in clearing the stand in a reasonably short time.

Constructionally, the stand can be described as a composite construction of structural steel and reinforced concrete. The roof (which has light loading and long span), the internal stanchions, and the floor and raking joists are of structural steel. Foundations, external columns, beams, terracing and floor slabs, are of reinforced concrete. The main columns supporting the roof are similar to those at the Stadium, Wembley, being of reinforced concrete giving the minimum obstruction to the spectators. Great regard has been paid to lessening obstruction to the spectators' view, and for this reason the concrete columns supporting the roof have been placed 81 ft. apart instead of the 30 ft. to 40 ft. which is common in football stands. The front of the roof cantilevers forward almost as far as the "touch line," the projection being 40 ft. The stand is glazed more than is usual, giving a very bright aspect.

The wind pressure from the roof is taken on the rear reinforced concrete columns. These rear columns are on the boundary of the property, and so as not to trespass on the adjoining land they are carried on a reinforced concrete strip foundation 2 ft. 6 in. wide. The roof of the demolished stand was re-erected over one of the existing standing terraces, giving weather protection to 6,500 spectators.

The stand was designed by Sir Owen Williams, K.B.E., B.Sc., M.Inst.C.E., the Engineer of the Wembley Stadium, and Mr. W. J. Cearns, of Stratford, was the general and structural steelwork contractor.

The photographs on p. 622 illustrate the completed stand.

The Secretary of the West Ham United Football Club.

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Book Reviews.

Reinforced Concrete in Practice. By A. Alban H. Scott. Second Edition.
[London: Scott, Greenwood & Son. Price 8s. 6d. net.]

To cover the whole ground of the practice of reinforced concrete in 188 small pages is a difficult task, and one could have hoped that the author of this book had given us more of his practical experience in actual constructional work. The 40 pages devoted to "Testing of Materials," mostly under laboratory conditions, and the pages outlining the processes of steel manufacture could, we think, with advantage have been condensed and more information given from the author's practical experience in carrying out reinforced concrete work. If in a future edition the author could see his way to give more practical hints on the execution of reinforced concrete construction the book would be of even more service to an industry not too well supplied with really practical literature.

Roof Coverings. By Ernest G. Blake.
[London: Chapman & Hall, Ltd. Price 10s. 6d. net.]

In this volume the author is nothing if not thorough, and not only gives the modern application of the various types of roofing materials, but also outlines their history. The book covers 260 pages, and deals with every roofing material now in use. In a well-produced book such as this one could have wished that the application of the various materials had been kept on more general lines, and that the index were not disfigured by names of a large number of privileged firms and trade names staring in heavier type than the rest; large parts of some of the chapters read very much like trade catalogues, and when, such as here, an effort is made to mention a large number of brands of patented articles and products there is a danger that some of equal, or perhaps greater, merit are not referred to at all. The book would be improved from the reader's point of view if these descriptions of special products were collated in a chapter by themselves, and indexed separately.

Books on Business and Economics.
London: McGraw-Hill Publishing Co., Ltd.

This is the title of a list issued by the McGraw-Hill Publishing Co., Ltd., containing particulars of the very large number of books they publish. The subjects cover a very wide range, and included therein are a host of books of value to any one engaged in business. Copies of the list will be sent post free on request.

Shuttering Competition.

The Committee appointed by the Minister of Health "to inquire and report as to new materials or methods of construction which are or may be available for the building of houses for the working classes and to make recommendations as to the organisation required for securing the adoption and use of approved new materials or methods by local authorities and other bodies or persons providing such houses," has now issued its report on the competition for an improved type of shuttering for the erection of concrete cottages. The primary object of the competition was to stimulate and bring out new ideas, and methods and systems already in use, or published by patents, were, therefore, specifically excluded from the competition.

The Committee is of opinion that the system submitted by Mr. R. L. Bendall, of "Scarborough," Richmond Wood Road, Bournemouth, is the best entry received, and, therefore, under the terms of the competition Mr. Bendall has been awarded the prize of £250. The Committee states, "While we are satisfied that this system has merits and is the best of those submitted to us, we think it necessary to state that it must not be regarded as superior to some of the systems which are already in use, or published, and were, therefore, ineligible for this competition. We have not felt justified in advising that the further prizes which it is within our discretion to recommend should be awarded."

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New Catholic Church at Totton.

THE new church at Totton is of special interest in that it is, we believe, the first church to be erected on a new system of construction evolved by Messrs. J. H. & W. C. Mangan, architects, of Preston.

The object in view has been to provide a small church in a permanent form of construction and possessing all the characteristics of form and appearance usual and desirable in a church building at a cost approximating closely to the cost of what are known as "temporary" or "semi-permanent" buildings. These latter, of which there are several types on the market, are usually of stereotyped design, and seldom possess any architectural character. Indeed the very nature of the materials used and the method of construction adopted make it almost impossible to produce the effects rightly looked for in a church, and which so far have been only obtained in carefully designed buildings of brick or stone.

At Totton, therefore, the architects, with the full co-operation and encouragement of the Bishop of Portsmouth, broke away from conventional usage and embarked on an experiment—the success of which should have far-reaching effects. Briefly described, the church is a rectangular building constructed with a steel frame, the whole of which, with the exception of the roof principals, is hidden from view. The walls are constructed of concrete slabs of a new type and having a continuous cavity throughout. Internally the wall slabs are finished to a Bath stone colour, the joints being pointed in white cement. Externally the lower parts of the walls are of red rusticated blocks, giving the effect of red sandstone, rising from a stone-coloured plinth; the upper part is of cream-coloured sand-faced blocks. The elevations are broken by suitably placed bands and buttresses, and pierced by circular-headed quoined windows of the colour and texture of Portland stone, and filled in with leaded glass in metal casements. The effect externally and internally is that of a stone-built church, and lacks entirely the drab monotony of colour and texture often associated with concrete buildings. Deep overhanging eaves and verges are

[*Messrs. J. H. & W. C. Mangan, Architects.*

New Concrete Church at Totton.

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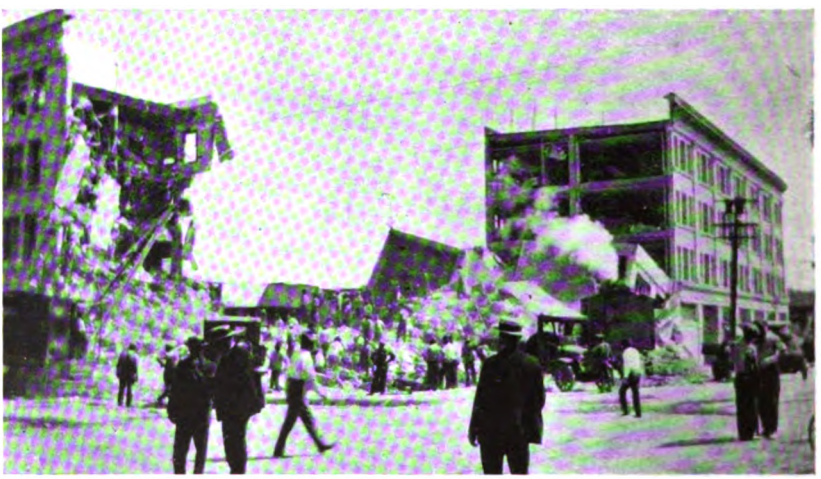
provided to the roof, which is hipped and covered with French tiles laid on boarding. Internally the roof is finished in fibrous plaster panels between the rafters.

At the west end an entrance porch is surmounted by a gabled niche, which will carry a Statue of St. Teresa of Lisieux—" The Little Flower," to whom the church is dedicated. So far as we know this will be the first completed church in Great Britain dedicated to the recently canonized Saint. The whole of the woodwork is of Columbian pine stained and varnished, and the floor is of concrete finished in a waterproof composition of a rich red colour. The altar has been specially designed and executed in artificial Caen stone with modelled plaster ornament and gold fabric panels.

Seating is provided for 108 people, but the design and method of construction adopted lends itself to extension without much material alteration to the original structure as the congregation at Totton increases. There is a Sacristy and cleaners' room, and a central heating system has been installed. No particular architectural style has been followed, but the general feeling is Romanesque. There can be no question that the stability and permanence of the structure are at least equal to that of a brick or stone building, and the cost of repairs and maintenance should be, if anything, rather less.

The church is built on a commanding corner site on the main road, and will be surrounded by a concrete palisade wall of novel design, with entrance gates and steps. The whole of the work has been carried out, under the architects, by Messrs. Marchetti, Ltd., of Copnor Road, Portsmouth, at a cost of approximately £1,250. Although carried out on a small scale this little church will go far to display the advantages of concrete construction when properly designed and handled.



A Reinforced Concrete Building (Right) after the Santa Barbara Earthquake.

On this and the next page we give some views taken in Santa Barbara immediately after the recent earthquake. As we stated in our September issue, the reinforced concrete buildings in the earthquake zone were practically undamaged, while those built of brick and stone were badly affected, as shown in the illustrations. We are indebted for the photographs to Mr. J. E. Wallis, of Rickmansworth, by whose son they were taken.

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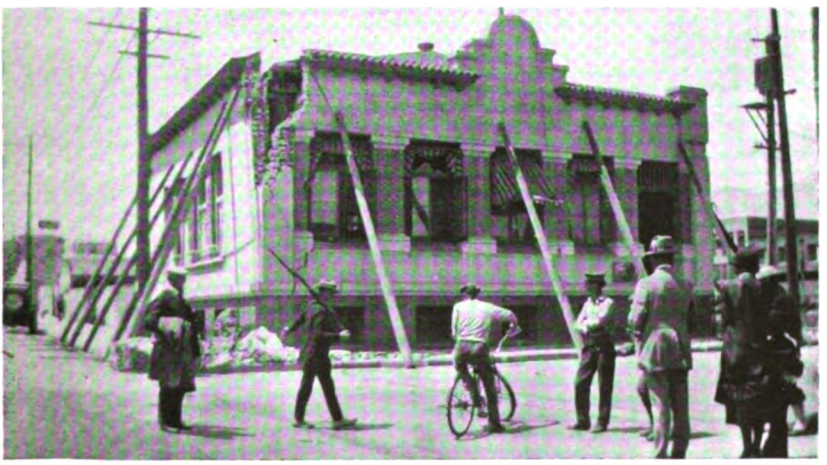

Earthquake Effects at Santa Barbara. (*See p.* 626.)

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Collaboration in Bridge Design.

THE new Clark's Ferry bridge over the Susquehanna River near Harrisburg, Pennsylvania, has recently been completed, and measures about half a mile from shore to shore. The roadway is 20 ft. wide. The twenty arches rise to a considerable height above the river surface, thus giving ample security against flood waters and ice.

The design and supervision was made by Mr. F. M. Masters with Mr. R. Modjeski, Chief Engineer of the Delaware River Bridge, as consultant, and Professor Paul P. Cret, University of Pennsylvania, as consulting architect.

Particular stress in the design of the bridge structure was laid upon the ornamentation of the roadway, and at the suggestion of the Pennsylvania State Art Commission, provision was made for a lay-out of light standards which were designed by Professor Cret. The Westinghouse Company was commissioned to execute the work, and a special Westinghouse lantern was adopted and incorporated in the design of each of the light bases for the respective arches and abutments at the bridge approaches. Construction of the bridge began in June, 1924, and completed in June this year, by the Vang Construction Co., and is said to be a record for this type of work. The installation of the illuminating fixtures was completed in August.

Clark's Ferry Bridge, Pennsylvania, U.S.A.

Design of Formwork for Reinforced Concrete Construction.

We regret that owing to pressure on our space we are unable this month to give Part XI of Mr. A. E. Wynn's series of articles on the "Design of Formwork for Reinforced Concrete Construction." We hope to continue these in our next issue.

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The All-Concrete Road.

WHAT are the qualities of the perfect road? First and foremost it must be capable of carrying the type of traffic for which it is built without breaking up. It must cost no more than is absolutely necessary to achieve this strength. When laid it must be durable. These are the primary requirements of a road, and no one will dispute that they are best met by reinforced concrete; where the weight and volume of traffic demand something more than a macadam foundation no other material is, in fact, ever considered. It is only when we come to the other qualities essential to the perfect road that differences of opinion exist. Not only must the perfect road be strong and durable, but it must also have a surface which is smooth yet not slippery; it must not be injured by changes in climatic conditions; the surface must be wear-resisting so as to avoid constant expenditure on repairs.

How does concrete fulfil these conditions? It is in regard to the first that the greatest conflict of opinion is to be found, and it is to be regretted that reasoned argument and practical experience are so frequently befogged by sweeping condemnations and generalities. We are told, for instance, that the naturally rough surface of concrete is slippery. Such statements do not call for discussion, but it is surprising how frequently they are made. Any motorist will tell us that the safest roads on a "greasy" day are the unsurfaced macadam roads, and the concrete surface is the nearest we can get to macadam consistent with durability. The statement of the London General Omnibus Company (given elsewhere in this issue) on the concrete-surface stopping-place at Wembley, in which they say not one of the half-million or so buses which have used it during the run of the Exhibition has skidded, is sufficient evidence of the absence of this danger on a concrete surface.

Of the other essentials of the perfect road, no surface is immune from injury by climatic conditions, unless it be granite setts. Concrete expands with heat and contracts with cold, and cracks appear unless proper precautions are taken. Until a material is discovered which is not affected by climatic conditions we have to choose the material which gives

Fairview Road, Dublin.

A Reinforced Concrete Road in Ireland.

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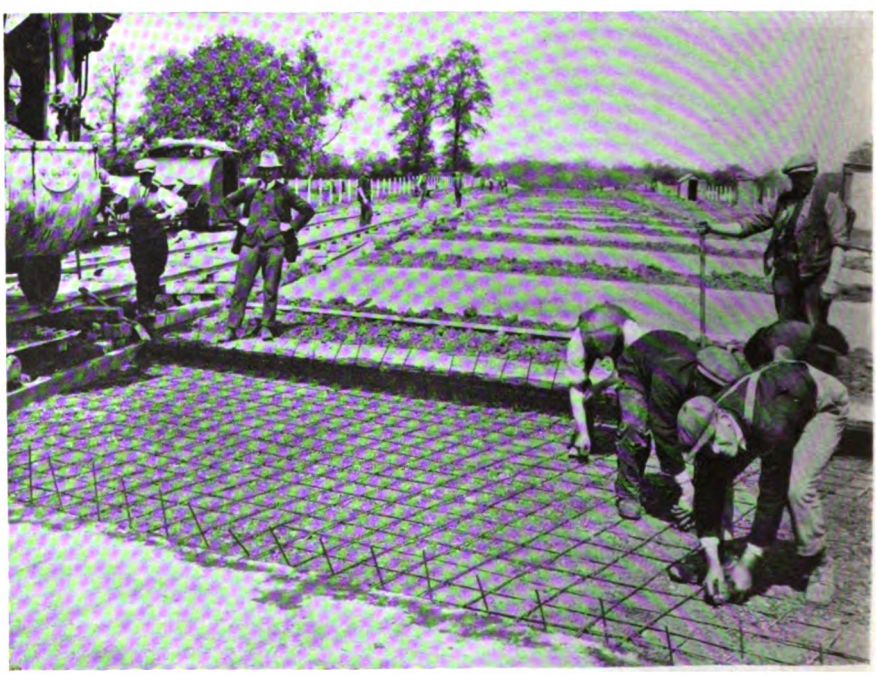
the least trouble from this cause. It must be admitted that many of the earlier all-concrete roads show decided cracks, but it is less than fifteen years since the first all-concrete road was laid in this country, and if some of the earlier ones have failed through lack of experience they have served a useful and indispensable purpose in providing the basis of the knowledge out of which have developed the roads of this type now being laid all over the country. With proper reinforcement, good materials and workmanship, there is no reason why serious cracks should occur in an all-concrete road, and in the roads laid recently they do not appear. The all-concrete arterial and by-pass roads in Surrey, those in Southwark and the Docks, and many others show no signs of anything but hair cracks after several years' wear.

The last essential is economy of maintenance, and here again there have been considerable developments within the last few years. The old trouble of dusting-up can be avoided by the application of silicate of soda, which hardens and waterproofs the surface so that traffic will not affect it for many years. This inexpensive treatment is being applied on most of the newer roads with eminently satisfactory results.

We may let facts and figures speak for themselves. The first all-concrete road in this country was laid in 1912. The war intervened and new road construction practically ceased, with the result that in 1919 there were only 25 such roads. But within three years this number had increased to 227, and to-day we can count no less than 600—and new all-concrete roads are being started almost every week. In the United States approximately 90 per cent. of the new roads undertaken are being built of concrete, and in that country there are now over 32,000 miles of this type of road. Considering the advantages of the concrete road in all the essentials which go to produce the perfect road, together with the low cost (in many districts they are being laid at round about 10*s*. a yard super), it is not surprising that Borough Engineers and Surveyors are taking such a keen interest in them, and that the interests which are affected by their use are loud in their denunciation. But progress is not to be denied, and we look forward confidently to a time not far distant when the all-concrete road will be the accepted type.



Sutton By-Pass, for the Surrey County Council. (*See p.* 631.)

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Concrete Roads in Surrey.

SOME of the best all-concrete roads in the country are those built in Surrey under the supervision of Mr. W. P. Robinson, A.M.Inst.C.E., M.I.M. & Cy. E., the County Surveyor. These arterial and by-pass roads generally consist of a carriageway of 30 ft. and one footpath 8 ft. wide only at present, thus providing ample width for present requirements and giving scope for extension up to a width of 100 ft. if demanded by the traffic of the future. The portion now laid is in the centre of the 100-ft. width which has been fenced in, and the remainder is to be kept as a grass verge until it is necessary to widen the paved portion.

The subsoil throughout the length of the road is mostly stiff sandy clay, which, after grading, is covered with a bed of clinker well rolled. The carriageway is laid entirely in reinforced concrete with a total thickness of 8 in., the lower course 6 in. thick and the top course 2 in. thick. The lower course is composed of 4 parts coarse material (60 per cent. by weight passing a 1-in. square mesh and rejected by ½-in. square mesh; 40 per cent. by weight passing ½-in. square mesh and rejected by ⅛-in. square mesh), 2 parts sand, and 1 part British Portland cement. The top, or wearing, course is made of 3 parts coarse material (60 per cent. by weight passing a ¾-in. square mesh, and rejected by a ⅜-in. square mesh; 40 per cent. by weight passing a ⅜-in. square mesh, and rejected by a ⅛-in. square mesh), 1½ parts sand, and 1 part Portland cement. The coarse material is obtained locally from the valley of the Thames, and is crushed, screened, washed and graded near the site. The sand is specified to conform to the following grading with a margin of 5 per cent.:—

	By Weight %
Passing 10-mesh sieve and rejected by 30-mesh sieve	55
Passing 30-mesh sieve and rejected by 80-mesh sieve	30
Passing 80-mesh sieve and rejected by 100-mesh sieve	15

The concrete is mixed by batch-type mixers with a capacity of 14 cu. ft. of mixed material, the loading skips having a capacity of 24½ cu. ft. The quantity of water used in mixing the concrete, which varies and depends upon atmospheric conditions and the quantity of water present in the sand and aggregate, is as near as practically possible an amount

The Reigate-Dorking All-Concrete Road.

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Section of Birmingham to Nuneaton Main Road, for the Warwickshire County Council. (*See p.* 643.)

Dunston Road, Shoreditch, for the Shoreditch Borough Council: Reinforced Concrete, 6 in. thick. (*See p.* 643.)

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War Office Road, Foulness: Artillery on a Concrete-Surface Road on a Wet Day. (*See p.* 643.)

equivalent to 25 per cent. by weight of the cement used in the mix, or between 2·5 per cent. and 3 per cent. of the total quantity of material used in the mix. When the sand and aggregate are saturated with water to such an extent that a slight "sweat" appears when the concrete is punned into position it is found it is not necessary to add any water to the mix. This is proving quite satisfactory; the concrete is absolutely dense and so firmly is it bound together that it is impossible to dislodge with a hammer the stones showing on the surface.

The concrete is laid in bays the full width of the road by 14 ft. in length, on the alternate bay system with the joints between the bays at an angle of 60 deg. from the kerb. In addition to reducing the liability of damage to the edges of the bays by ensuring that traffic will strike them at a more acute or obtuse angle than a right angle, and also to obviate two-wheel loads of any vehicle striking the joint at the same time, thus reducing the stress along the bay, it is also anticipated that this method of laying the bays at an angle of 60 deg. will reduce the resistance of the concrete to expansion and contraction, and thus assist in eliminating any tendency to bulge and cracks due to expansion.

In order to ensure that the two layers of concrete are laid to the specified depth, a simple method of screeding is adopted. Before placing any of the concrete, bolts are driven into the prepared sub-grade where the edges of the bay will come, one at each side of the road and one in the middle. A board six inches in depth, with holes already bored, is then slotted edgewise on the bolts, thus forming the shuttering for the edges of the lower course. After the first course is laid to the level of these boards, other boards, two inches in depth, are slotted on the bolts in the same way, forming the shuttering for the edges of the top course and a screed for levelling the surface of the road.

The reinforcement is wired up into mats on the site, and placed top and bottom of the road; that is, 1½ in. from the bottom and 1½ in. from the top. In order to ensure equal settlement of contiguous bays in the event of settlement taking place the reinforcement is left projecting about 1 ft. from the edges of the bays laid first, and is kept in position by passing the longitudinal bars through holes made for the purpose in the transverse side forms. The top course in each bay is laid immediately after the lower course in order that there shall be no danger of bad adhesion owing to the setting of the lower course.

Within 24 hours of the concrete being placed the surface is brushed with stiff brooms to remove all scum, and is then immediately covered with a layer, 3 in. thick, of soil, which is kept damp with water for at least 28 days.

A period of not less than 14 days is allowed before the intermediate bays are concreted.

The contractors are Messrs. Stewart & McDonnell, and the aggregate is being supplied by the Ham River Grit Co., Ltd., no less than 400 tons a day being delivered on the Kingston by-pass.

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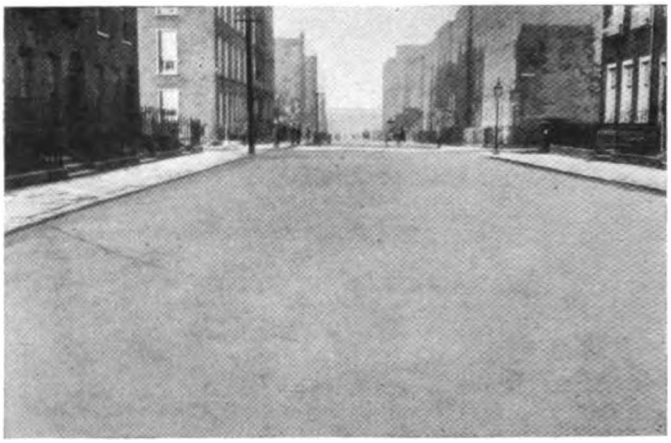

An All-Concrete Road at Limerick. (*See p.* 635.)

Wembley's Non-skid Roadway.

EVIDENCE of the non-skid properties of concrete surfaces is furnished by figures compiled at the omnibus station at Wembley. These show that the concrete-surfaced 'bus station had carried, without an accident, about one million buses and char-a-bancs, besides many thousands of taxicabs, vans, and private cars, by the time the Exhibition closed its gates this season. The area, which measures 6,500 square yards, and occupies a decided slope, serves the main entrance to the Exhibition grounds; and, according to the London General Omnibus Company, not a single case of skidding has occurred since the roadway was laid in April last year. During the Exhibition twelve omnibus services used the station, and between 10 a.m. and midnight throughout the week the arrivals averaged one 'bus every half-minute. The concrete paving, which is 9 in. thick and reinforced by B.R.C. fabric, was laid by the Omnibus Company's staff.

Wembley Omnibus Station.

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Reinforced Concrete Road at Tilbury.

CALCUTTA Road, illustrated on this page, is one of the many all-concrete roads laid at Tilbury under the supervision of Mr. S. A. Hill-Willis, Engineer and Surveyor to the Urban District Council. The specification of this particular road called for a lower course of 4½ in. of 4 : 1 concrete and a top course of 1½ in. consisting of 3 parts best granite chippings evenly graded from ⅛ in. to ½ in., 1½ parts sand, and 1 part Portland cement; the aggregate, chippings and sand for the top course not to have voids exceeding 20 per cent. The reinforcement selected was "Maxweld," Ref. No. 5B Fabric, which is made from hard-drawn steel of high elastic limit and weighs 4·7 lbs. per yard super.

Reinforced Concrete Roads at Limerick.

In the repaving works of the Limerick Borough Council, generally the whole of the roads are 7 in. thick, laid with a bottom course of 5 in. 7½ : 1 concrete, and a top course of 2 in. of 3 : 1 concrete, reinforced with "Maxweld" double layer fabric, weighing 10 lbs. per yard super, manufactured by Richard Hill & Co., Ltd., Middlesbrough. The works were carried out, under the direction of Mr. J. J. Peacocke, by Mr. P. Dillon, of Limerick.

Some of these roads are constructed over cellars which run under the roadway and are divided by a party wall in the centre. With the advent of heavy traffic the brick arches began to deteriorate seriously, and the engineer was faced with the problem of either filling in the cellars or, as an alternative, constructing a concrete slab over the whole of the street, the latter construction being adopted with remarkable success.

These roads are laid on the continuous system, i.e. at the finish of a day's work the concrete is finished to a straight edge, cleaned and washed the following morning, and continued again with a vertical joint, thus forming a monolithic structure through the whole of the road. The surface was treated with silicate of soda, and after fifteen months' wear the Engineer reports that the roadways are entirely free from cracks, and the surface shows no sign of wearing. This type of construction is now being extended, and a further extension has now been let for an additional 47,000 yards super of roadway.

Calcutta Road, Tilbury.

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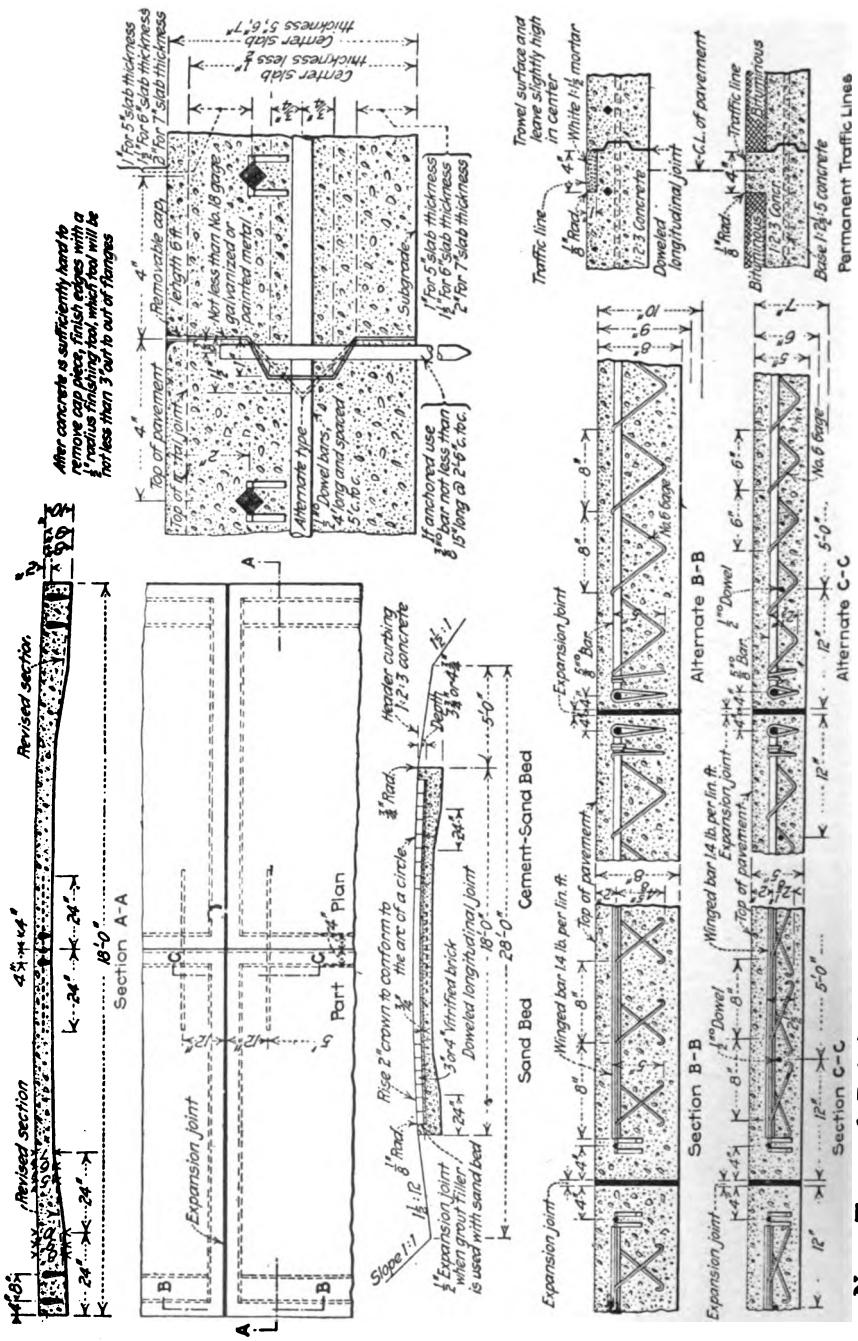

New Type of Reinforced Concrete Road Construction: Showing Method of Inserting White Traffic Lines. (*See p.* 637.)

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New Type of Concrete Road Construction.

On p. 636 we illustrate a new type of concrete road construction now in use by the Pennsylvania Highway Dept., and the smaller volumes required by this design are stated to effect a saving of £750 a mile.

As indicated by the drawings, the original design provided for a uniform thickness of 10 in. for the 2 ft. nearest the edge and then a gradual thinning down to the middle of the slab. This was rather the favoured section structurally, but the belief of the contractors that an unbroken slope from the edge in 4 ft. would be more simple to construct led to the decision to change future plans as indicated by the broken lines

Except in width of thickened edge zone, the chief structural departure from previous practice is the anchored edge and joint bars. It is considered that this design will add to both stiffness and bond, and also make the position of the bar more certain. Assembled into units before placing it is considered that the reinforcement can be quickly installed at a saving in cost over full-slab-width mesh reinforcement.

White Lines.—An innovation in design of considerable interest to road engineers in this country just now is the permanent traffic lines indicated by the enlarged detail on the accompanying drawing. The specification for this line is as follows: Traffic lines shall be composed of 1 part white cement and 1½ parts of washed sand, mixed thoroughly to a stiff consistency. They shall be constructed in the centre of the width of the pavement on curves, and extended on tangents as required. They shall be 4 in. in width and 1 in. in depth unless otherwise specified. The recess for placing this traffic line shall be formed by placing a strip of wood 1 in. in depth and 4 in. in width with the edges chamfered slightly, in the correct position when the concrete is placed and finished. Where the recess is formed with a wooden strip, the wooden strip shall be cut in segments, not more than 2 ft. in length, and attached with screws to a light metal strip on one edge, so that it will conform to the curvature satisfactorily. The traffic line shall be constructed from four to twelve hours after the surface of the pavement has been finished, or as soon as the workmen may stand upon the surface of the finished pavement without marring it. After the form has been removed the concrete shall be thoroughly soaked with water, and the white mortar shall be placed in the recess, tamped thoroughly in place, and finished with a trowel to the section shown on the drawing. The edges of the traffic lines shall stand up boldly and be rounded slightly as shown on the drawings.

Main Road between Chester and New Ferry, for the Cheshire County Council.
(*See p.* 643.)

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Road at Portslade Gas Works.

Road at Portslade Gas Works. (*See p.* 639.)

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North-Eastern Ring Road, Hull, for the Hull Corporation. (*See p.* 643.)

Road at Portslade Gas Works.

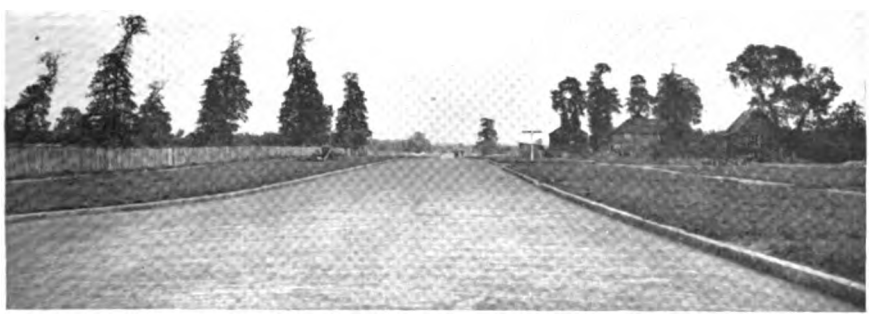
THE roads in the works of a large industrial concern are in the majority of cases subjected to traffic of an exceptionally heavy nature, and it is therefore not surprising that reinforced concrete has for many years past been very extensively used for the construction of such roads.

The reinforced concrete road for the Brighton and Hove General Gas Company was constructed last summer. It gives access to the Portslade Works of the Company as well as to the neighbouring Electricity Station belonging to the Brighton Corporation, and carries a heavy traffic of coke, breeze, etc., which is naturally effected by vehicles of a very heavy type.

Various methods of construction had been previously tried by the Company but none of them had proved entirely satisfactory, and it was, therefore, finally decided to adopt reinforced concrete. The road has now been in use for over a year and the result to date has been very satisfactory; it is in excellent condition and no expenditure on upkeep has been necessary. It runs along the banks of Shoreham Harbour and is nearly half a mile long and from 25 to 30 ft. wide, the width extending to the retaining wall adjoining Shoreham Harbour to which the water reaches at high tide.

The road consists of reinforced concrete 7 in. in thickness, the reinforcement consisting of double layer "Barmesh" weighing 9 lbs. per square yard which was fabricated on the site of the work by the Indented Bar Co.

The work was carried out under the personal supervision of Mr. H. V. Corfield, Assistant Engineer to the Brighton and Hove General Gas Co., at Portslade. The work was executed by the Gas Company's own staff at a cost of under 10*s*. per yard super.



An All-Concrete Section of the Great West Road: Reinforced Concrete, 8 in. thick. (*See p.* 643.)

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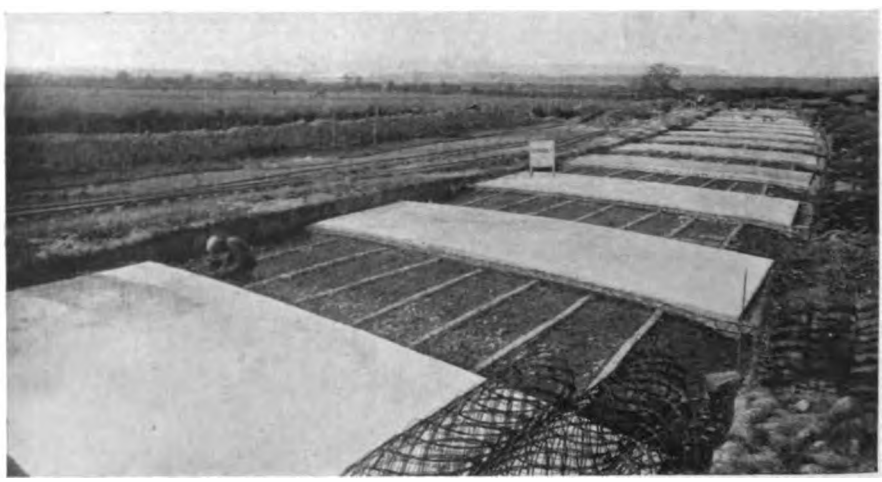
Morton to Middlesbrough Road, for the Yorkshire County Council: Reinforced Concrete, 9 in. thick. (*See p.* 643.)



Chester Ring Road, for the Chester Corporation: Reinforced Concrete, 8 in. thick. (*See p.* 643.)

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Roads for Heavy Traffic.

A METHOD of construction which they claim affords a perfect and lasting surface for fast and heavy traffic has been adopted by the Walker-Weston Co., Ltd. This surface is known as "Macrete," and is laid with free maintenance guarantees of five years and upwards. The photographs on the next page show roads of this type being constructed for the Port of London Authority at the Surrey Commercial Dock under the direction of Sir Edwin Cooper, F.R.I.B.A.

The chief features differing from ordinary forms of road construction are :—

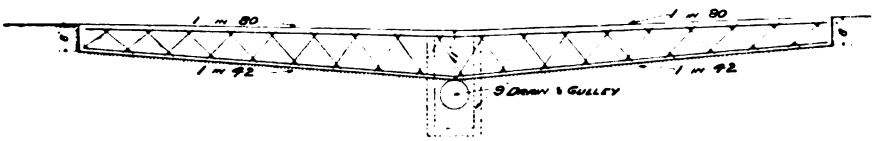
(1) Inverted section, as shown in the drawing on this page, with central channel and gulleys ;

(2) Concrete surface composed of large granite aggregate.

The road crust is composed entirely of concrete, the thickness in this case being 8 in. at the sides and 10 in. in the centre. The transverse slopes of the surface are 1 in 80, and the longitudinal fall between gulleys 1 in 150. By providing increased thickness at the centre of the road the formation slopes are greater than at the surface. This is designed to give maximum strength to combat longitudinal compression due to expansion, and to provide for lateral expansion by enabling the great compressive strength of the concrete to assist expansion up the formation, whilst it recognises the negligible tensile strength of the concrete by affording facility, with the aid of reinforcement, for contraction down the inverted formation slopes towards the centre of the road. Where the road is constructed in two halves, the central longitudinal joint is in compression due to gravity and the vibration of traffic. It is claimed that the tendency, observed in cambered roads, for the concrete to form a longitudinal crack at the crown of the road does not arise in this case.

One of the factors in the case of the road illustrated, leading to the adoption of an inverted section, was the fact that the adjacent ground will be used for the storage of stacks of valuable timber. The central drainage is calculated to reduce mud-splashing to a minimum, and although the particular consideration is not one of general import this characteristic of inverted roads is worthy of general attention in its relation to the protection of buildings and pedestrians. In the case of a cambered surface all traffic has a tendency to use the crown of the road, but it is contended that with an inverted surface with flat slopes every inducement is offered to traffic to use the whole surface, and fast over-taking traffic on the off-side of the road is provided with a surface suitably banked.

The concrete surface of the road consists of a 2½-in. layer of large granite macadam, sand and cement, mixed in the proportions 3 : 1¼ : 1, superimposed on a base of 6 to 1 ballast concrete. The concrete is laid in alternate bays, each 10 ft. wide, and is reinforced with Walker-Weston patent Pyramidal reinforcement, weighing 11 lbs. and 14 lbs. per sq. yd. respectively, for the 30-ft. and 50-ft. wide roads. No steel is carried through the joint between adjacent bays. These are treated as expansion joints, interlocked by means of a patent joint.



"Inverted" Road Construction.

The concrete surface has excellent wearing qualities, and provides a perfect grip for horse-drawn and motor traffic. Being composed of large aggregate, the surface is, in effect, a development of a macadam road, and is admirably adapted for receiving and affording perfect adhesion for bituminous dressings sprinkled with granite chippings. Such carpets, under exacting conditions of main-road dock traffic, have lasted in this condition for over four years. Owing to the great compressive strength of the concrete crust under all weather conditions, the surface

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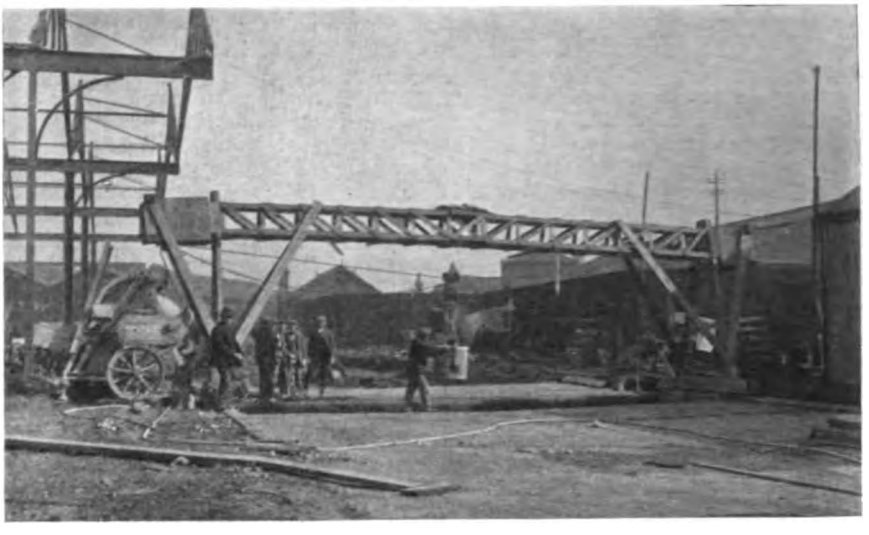

Laying Concrete Roads at the Surrey Commercial Docks.

does not corrugate, but offers resistance at all times to the destructive action of fast and heavy traffic.

The roads illustrated are practically straight, but the design lends itself for transition at curves and bends into banked slopes, and has received the sanction of the Ministry of Transport.

A feature of the process of construction is the patent overhead strut cableway used for concreting, shown in the photograph. The essential principle of this plant is that the load is supported on a wheeled carrier, traversing a cableway suspended beneath a light girder. The pull of the cableway rope tends to buckle the centre of the overhead girder upwards, and thus to neutralise the sagging effect induced in the girder by its own weight. By this arrangement the girder merely acts as a strut to keep the ends of the cableway apart. The whole plant as shown was constructed by unskilled workmen at a total cost of under £50.

An All-Concrete Road at the Surrey Commercial Docks.

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A Test Load of 55 tons on a 32-ft. Concrete Road at Tilbury.

Concrete Roads in London.

FOLLOWING the example of the neighbouring borough of Southwark, where Mr. Arthur Harrison, the Borough Engineer, has been laying all-concrete roads for several years past, some roads of the same type are now being laid in Camberwell under the direction of Mr. William Bell, the Borough Engineer and Surveyor.

In a paper read before the Institution of Municipal and County Engineers last month, Mr. Bell expressed some interesting opinions on concrete road construction. On the question of whether it is cheaper to lay a greater thickness of plain concrete or to use a thinner slab with reinforcement, he is in favour of the latter course. He has found that the cost of single-layer reinforcement is equivalent to about 1½ in. of concrete, and prefers a percentage of 0·4 per cent. of reinforcement if it be mild steel or 0·25 per cent. in the case of drawn steel. As to the provision of expansion joints, although he has seen many all-concrete roads laid without expansion joints and in good condition he favours the provision of such joints as a precaution against the appearance of cracks.

The usual concrete used at Camberwell is composed of 1 part Portland cement to 6 parts of Thames ballast, but for the best results the author recommends a specially graded mixture of 1 part Portland cement to 4 parts of ballast passing a 2½-in. mesh and retained on a 1-in. mesh to 2 parts sand. On completion the whole surface is treated three times with silicate of soda, which the author claims to be "much better than any tar-spraying."

Our Illustrations.

OF the roads illustrated in the preceding pages, The British Reinforced Concrete Engineering Co., Ltd., supplied the reinforcement for the section of the Gt. West Road; the North Eastern Ring Road; the Omnibus Station at Wembley; the War Office Road at Foulness; Dunston Road, Shoreditch; the Birmingham to Nuneaton Main Road; and the road at Tilbury illustrated on this page. BRC. Fabric has been used to reinforce a very large number of all-concrete roads, including those at Motherwell, which, laid in 1923, are now stated by the Borough Surveyor to "have exceeded expectations"; at Ipswich, laid in 1923, and now reported by the Borough Surveyor to be "as good as the day when they were laid"; at Rhyl, which the Borough Surveyor reports as having cost nothing for maintenance; at Middlesbrough, which the Borough Surveyor states are "very satisfactory, have cost nothing for maintenance, and further streets will be laid with the same material in future"; and at Portsmouth, which have so far cost nothing for maintenance.

The Expanded Metal Co., Ltd., were responsible for the supply of the reinforcement for the Main Road between Chester and New Ferry; the Chester Ring Road; and the Morton to Middlesbrough Road.

Messrs. David Weston & Co., Ltd.'s, alternate bay system, and a junction on which it has recently been used, are illustrated on p. 644.

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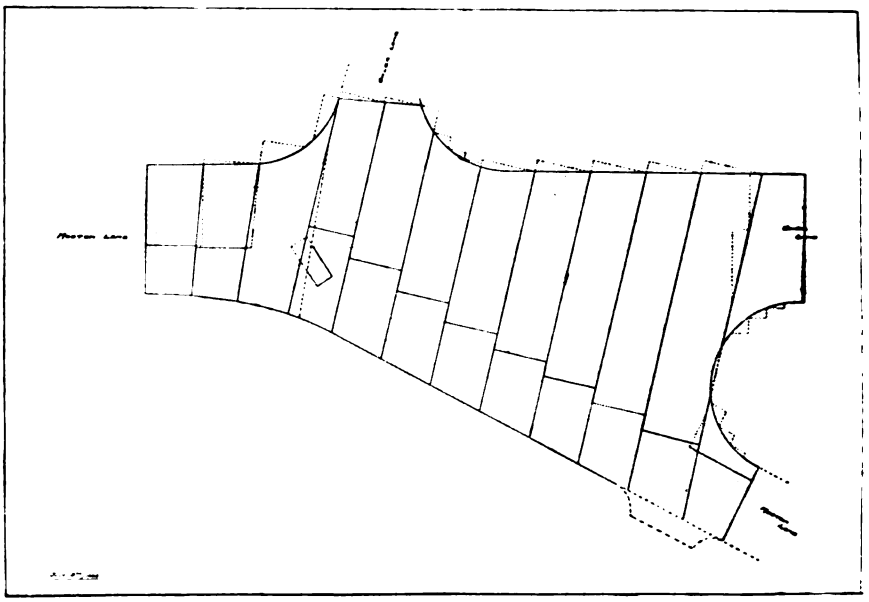
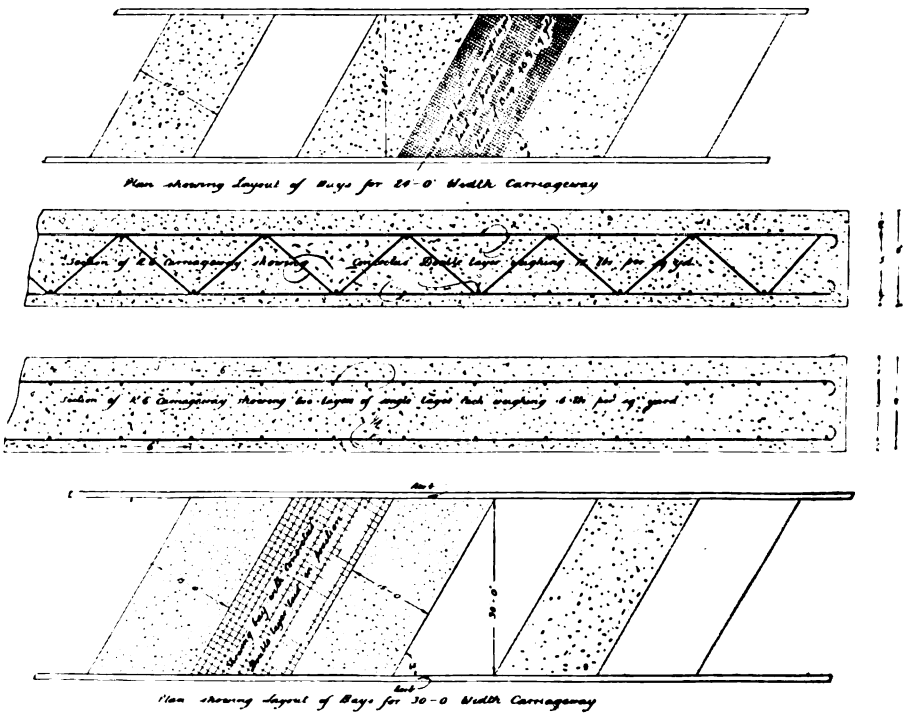


Plan Showing Lay-out of Reinforcement at an Awkward Junction.
(*See p.* 643.)



Method of Laying Double-Layer Reinforcement on the Alternate Bay System.
(*See p.* 643.)

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The Tirso Bridge, near Tadasuni, Sardinia.

One of the most interesting hydraulic undertakings of recent years is the Tirso project for the utilisation of the very variable flow of this river by means of a dam which stores 15,600,000,000 cu. ft. of water. The objects of this undertaking are the provision of water-supply and power and the prevention of malaria. The height of the dam across the valley is no less than 197 ft.

In a recent number of *Beton u. Eisen* is given a description of a noteworthy feature of the undertaking, namely, the new reinforced concrete bridge, about 6 miles from the dam, which was constructed to replace the old iron lattice bridge on the Abbasanta—So gono road. This old bridge is now submerged. Originally it was intended to erect the dam itself at this site whereby the construction of a new bridge could have been avoided, but for various reasons this plan was abandoned. If it had not been for the poor quality of the foundations on the sides of the valley the bridge would have been constructed as an arch of about 400 ft. span with trestle framed approaches at both ends of the arch. In the river bed the sub-soil is sound rock of good quality, on the right bank it is clay and trachyte, and on the left bank volcanic tufa. A rigid frame design with many columns was decided upon. The whole length of the bridge is 935 ft., divided into twenty-one equal spans. The overall width is 21 ft. 6 in., including a 15 ft. 9 in. roadway which is 160 ft. above the upper surface of the foundation to the highest trestle. The roadway is carried by five main girders each 10 in. wide; at mid-span the outer girders are 52½ in. deep and the inner girders 46 in. deep; over the supports the depths of the girders are 75½ in. and 72 in. respectively.

The columns of the ordinary trestles are of channel section, varying from 51 in. by 41½ in. to 39½ in. by 41½ in. The longitudinal stiffeners between each pair of trestles are 13·8 in. wide and from 31·5 to 55 in. in depth. The trestles themselves are stiffened transversely by cross-waling pieces, the upper wales being 15·8 in. by 11·8 in. and the lower 11·8 in. by 23·6 in. deep. Between the walings diagonal bracing was introduced.

According to the original design the

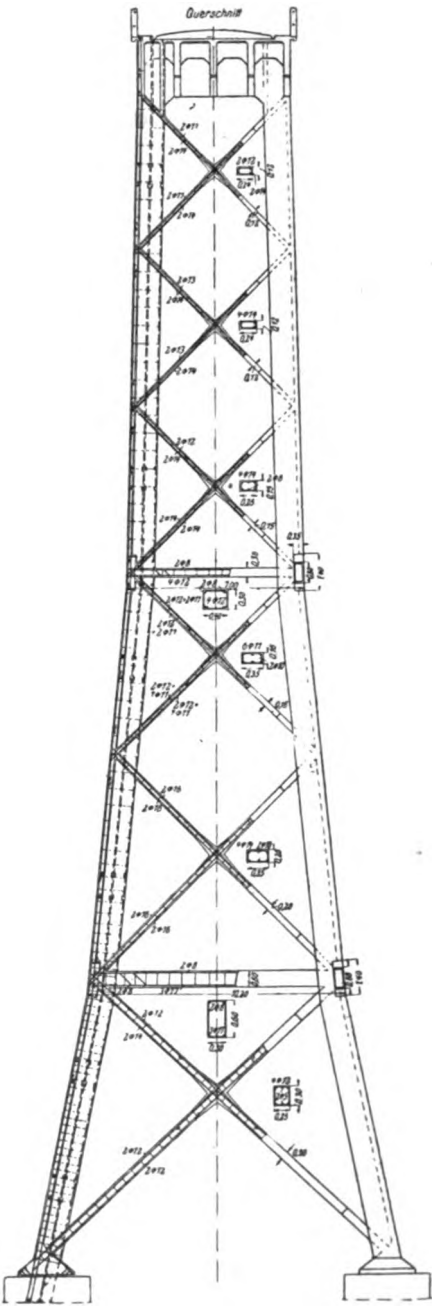

Fig. 1.

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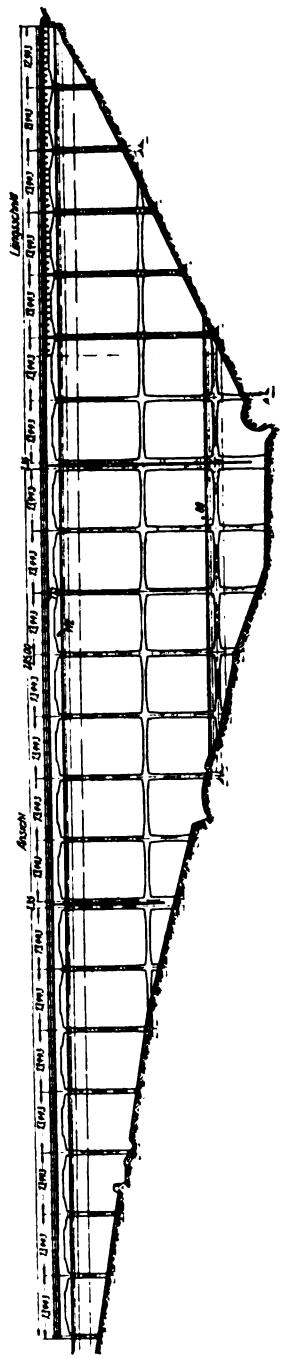

Fig. 2.

bridge was to be divided by two double columns and expansion joints into three equal lengths of seven spans each. During the construction the original design was altered and specially designed trestles, stronger and stiffer than those supporting the other spans, were built at the ends of the sixth span from the left bank of the gorge. At these points expansion joints were inserted in the main beams, and the longitudinal walings were omitted from this span.

For stress calculations the loadings used were (*a*) uniformly distributed load of 82 lbs. per sq. ft.; (*b*) 15 ton roller and 3 wagons; (*c*) wind load of 30·7 lbs. per sq. ft.; (*d*) temperature variation of $\pm 15^{\circ}$C.

Great difficulties had to be contended with during construction. The majority of the skilled workers brought over from the mainland were stricken with malaria, and in the summer of 1921 the work had to be suspended owing to the severity of the disease. The transport of material from the railway station at Abbasanta was accomplished by teams of oxen. A total of 840 tons of cement, 3,200 yards of concrete and 151 tons of steel were used on the contract, corresponding to 4·13 lbs. of steel per cu. ft. of concrete. The total cost was 2,800,000 lire, equivalent to £112,000 at the normal rate of exchange.

Work was started in December 1920, and tests made in October 1923. Up to the end of 1922 the work was carried on under G. Luzzati, of the firm of A. Garboli, Rome, and from that date onwards until completion construction was by the "Società Forze Idrauliche ed Elettriche del Tirso," under the direction of Prof. Pozzo of Turin.

Fig. 2 is an elevation and part longitudinal section of the original design, while *Fig.* 1 shows the reinforcement of a trestle.—W.S.G.

SILICATE CLAY DEPOSIT.

ENGLISH COMPANY WITH VERY EXTENSIVE AND VALUABLE DEPOSITS OF SILICATE CLAY situated in British Columbia desire to negotiate for sale of same, or arrange finance for working. Railway and transport facilities excellent. Write H. C., c/o C. J. Walker's Advt. Agency, 24 Coleman Street, London, E.C.2.

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Questions and Answers relating to Reinforced Concrete.

Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.—ED.

Stress on Concrete.

QUESTIONS.—(1) The formula for depth of 1 : 2 : 4 concrete with compression of concrete taken as 600 lbs. and tension on steel as 16,000 lbs., is 95 bd^2, steel ·00675, n as 36 d. I am anxious to get the formula for depth, value of n, and amount of steel for 1 : 3 : 6 mixture and 1 : 4 : 8 mixture. Can you help me?

(2) In non-reinforced concrete slabs, what formula can I take for depth for 1 : 2 : 4, 1 : 3 : 6, and 1 : 4 : 8 concrete? I know the value of tension in the non-reinforced slab is generally taken as nil, but in concrete slabs fixed between girders shown in steel catalogues apparently the slabs are not reinforced and therefore some strength value is allowed for them.

H. M. P.

ANSWER.—If the safe stress on 1 : 3 : 6 concrete is taken as 450 lbs. per sq. inch, and the safe stress on 1 : 4 : 8 concrete is taken as 300 lbs. per sq. inch, then the figures for 1 : 3 : 6 concrete are:

Depth of neutral axis, 0·297 d
Percentage of steel, 0·418
Resisting moment, 60·2 bd^2

The corresponding figures for the 1 : 4 : 8 concrete would be

Depth of neutral axis, 0·22 d
Percentage of steel, 0·206
Resisting moment, 30·6 bd^2

In the case of non-reinforced slabs, ignoring the arching action of the concrete, the safe resisting moment in all cases is given by $\frac{fdb^2}{6}$ where f is the safe stress in the concrete in tension. This may be taken as 60 for 1 : 2 : 4 concrete, 45 for 1 : 3 : 6, and 30 for 1 : 4 : 8. In many cases arching action assists considerably, but these cases cannot be generally treated, and must be treated on their merits.

Silo Design.

QUESTION.—What are the principal points to be noticed in the design of a reinforced concrete silo 50 ft. by 15 ft. diameter for the storage of cut grass, etc.? I have calculated for the stresses due to longitudinal bursting by the formula $F = \frac{pd}{2}$ and to find p have assumed $w = 62{\cdot}4$ lbs. per cu. ft. Is this correct?

P. W. C. S.

ANSWER.—The design of the silo will call for the consideration of various points, among which are the following:

(*a*) the weight of the material to be stored,
(*b*) the angle of repose of the material,
(*c*) the friction of the material against the sides,
(*d*) the moisture content, and
(*e*) the situation of the silo.

The weight of all ordinary materials can be found in various handbooks, but freshly-cut grass is not given among these items. This material will produce liquid, the quantity varying according to the condition of the crop when put in, but it may reach as high as 10 per cent. of the total contents of the silo, and the weight of the material per foot cube is a difficult figure to state definitely. The assumed weight of 62·4 lbs. per foot cube given by the querist is undoubtedly on the high side; a weight of 53 lb. would probably be sufficient, and this allowance would cover any other ordinary crops that might be stored in the silo.

The situation is important as affecting the allowance to be made for wind pressure when working out the design.

The design of a deep silo is somewhat complicated, and the querist is advised to submit his drawings to an expert for approval unless he is quite conversant with the method of calculation.

A. L.

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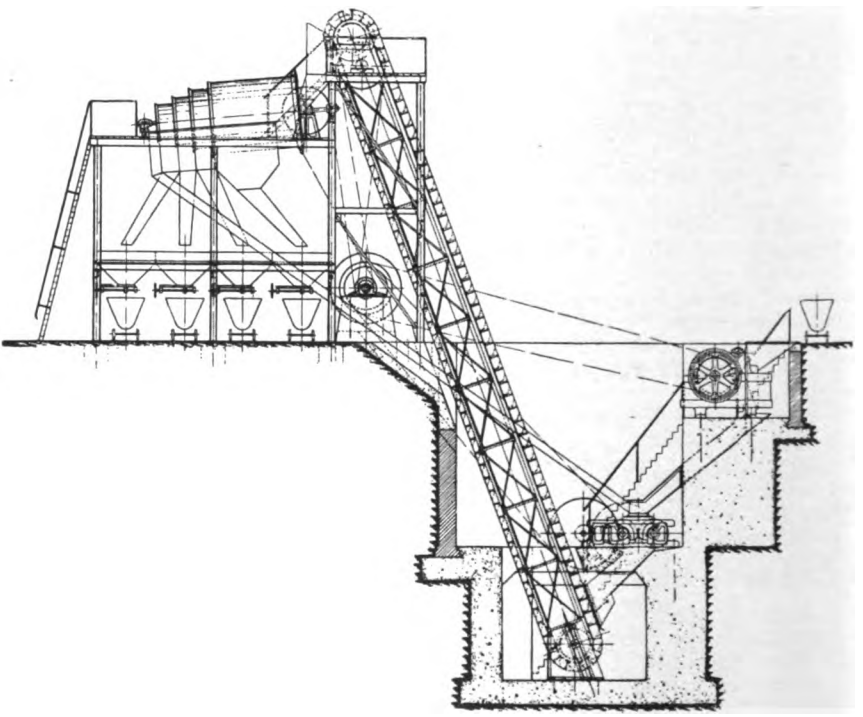
A Large Stone-Crushing Plant.

THE stone-crushing and screening plant illustrated on this page has been supplied by Messrs. Edgar Allen & Co., Ltd., of Sheffield, to the Calcutta Improvement Trust for crushing the enormous quantities of road metal required in the vast improvement schemes now being undertaken. The plant consists of four 24 in. by 14 in. Stag sectional type jaw crushers with cast steel bodies, and fitted with manganese steel jaw faces. Four 30 in. by 30 in. Stag cubing rolls cube the stone after leaving the jaw crushers. There are four telescopic screens for grading the stone into the various sizes, with hoppers, shoots, connections and shafting. The power plant comprises two tandem compound condensing steam engines, with piston valves of 150 I.H.P. each, and 130 R.P.M. The steam pressure is 145 lbs. per sq. in. The cylinders are 10 in. and 17 in. diameter by 24 in. stroke. Flywheels measure 8 ft. diameter by 19 in. free for belt drive. Steam is supplied by a Lancashire boiler 8 ft. diameter by 30 ft. (150 lbs. working pressure).

The complete crushing plant is designed to deal with approximately 80 tons per hour.

Serious transport difficulties were experienced in erection because nearly everything had either to be carried by coolies or transported in bullock carts across country. The fly-wheel of the engine, for instance, was actually rolled the whole way from the nearest point of the railway to the quarries.



A Large Stone-Crushing and Screening Plant.

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Thames ballast	,,	10	6
Broken brick (¾ in.)	,,	11	6
Best British Portland Cement	per ton 58s. to	63	6
"Ferrocrete" Rapid-Hardening Portland Cement	delivered London 10s. per ton extra		
"Super-Cement"	per ton 88s. to	93	0
"Lightning" Brand Aluminous Cement	,, 108s. to	113	6
Ciment Fondu	130/- ex stores		

BOARDING FOR SHUTTERING—		Sawn. s. d.	Wrot. s. d.
1 in.	per square	23 6	27 6
1¼ in.	,,	29 6	33 6
1½ in.	,,	35 6	41 0

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in. . . . . from £23 per standard
3 in. by 6 in. and 3 in. by 7 in. . . . . ,, £23 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅜ in. to 2⅞ in. Rounds	per cwt.	11	0
9/16 in. to ½ in. Rounds	,,	11	3
⅜ in. Rounds	,,	11	6
¼ in. Rounds	,,	12	6

Breeze Slabs per yd. super: 2 in., 1/11; 2½ in., 2/4; 3 in., 2/9; 4 in., 3/6.

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(*Based on Contracts up to £2,000.*)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 : 2 : 4—		s.	d.
Do. do. in foundation	per yard cube	48	6
Do. do. in columns	,, ,,	53	6
Do. do. in beams	,, ,,	53	6
Do. do. in floor slabs 4 in. thick	per yard super	5	8
Do. do. in floor slabs 6 in. thick	,, ,,	8	5
Do. do. in floor slabs 9 in. thick	,, ,,	12	3
Do. do. in walls 6 in. thick	,, ,,	8	8

(*Add for hoisting 3s. 6d. per yard cube above ground-floor level.*)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—		s.	d.
From ¼ in. to ⅜ in.	per cwt.	23	0
,, 9/16 in. to ½ in.	,,	22	0
,, ⅝ in. to 2⅞ in.	,,	21	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1¼*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2¼*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.* (per bend per cwt.).

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.* (per bend per cwt.).

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	,, ,,	1	1½
Do. do. to sides and soffits of beams, average 9 in. by 12 in.	,, ,,	1	1½
Do. do. as last, in narrow widths	,, ,,	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Carpenters working on old shuttering, 1/10½; Labourers on building works, 1/4½; Men on mixers and hoists, 1/5½; Bar-benders, 1/5½.

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Prospective New Concrete Work.

ABINGDON.—*Houses.*—The R.D.C. proposes to build 40 houses.

AYLESBURY. — *Houses.* — The R.D.C. proposes to build 100 houses.

BACUP.—*Reservoir.*—The T.C. has instructed the Borough Surveyor to prepare plans for the construction of a reservoir to hold 2,000,000 gallons.

BARNSLEY.—*Bridge.*—The Corporation has applied for a loan towards the construction of the Beevor Bridge.

BEDWELLTY.—*Reservoir.*—The U.D.C. has approached the M.H. for sanction to construct a reservoir in the Sirhowy Valley.

BIGGLESWADE.—*Houses.*—The U.D.C. is to build 41 houses on the Townfield estate. Mr. C. D. Woodward, of Biggleswade, is the contractor.

BISHOP AUCKLAND. — *Bridge.* — The U.D.C. has approached the London & North Eastern Railway with a view to erecting a footbridge at Grey Street.

BLACKWELL.—*Houses.*—The R.D.C. has applied for a loan of £50,000 for housing purposes.

BLACKWOOD.—*Houses.*—The Bedwellty U.D.C. is considering a scheme for the construction of 50 houses at Blackwood.

BURNLEY.—*Footbridge.*—The T.C. has instructed the Borough Engineer to prepare plans for the construction of a footbridge over the L.M. & S. Railway at Peace Street.

CHELMSFORD.—*Houses.*—The T.C. is considering a scheme for the erection of 250 houses.

CHESTER.—*Houses.*—The T.C. has applied to the M.H. for a loan of £49,164 for the erection of 102 houses.

CLACTON-ON-SEA.—*Pier.*—The scheme for the improvement of the pier is being considered by the Council.

COLCHESTER. — *Concrete Road.* — The Corporation has applied for a loan of £1,200 for constructing in reinforced concrete the approach to the sewage outfall works.

COVENTRY.—*Houses.*—The Corporation has asked for a loan of £270,000 for housing purposes.

DARLINGTON.—*Houses.*—The T.C. has applied for permission to erect 100 houses on the Eastbourne site.

DEWSBURY.—*Houses.*—The Corporation proposes to purchase 10 acres of land for housing purposes.

DUDLEY.—*Houses.*—Application has been made for a loan of £22,567 for the construction of 50 houses.

DURHAM.—*Houses.*—The R.D.C. proposes to erect 50 houses by direct labour.

EBBW VALE.—*Houses.*—The U.D.C. is considering a scheme for the construction of 100 houses.

ERSDON.—*Houses.*—The U.D.C. is considering a scheme for the erection of 100 houses.

ESSEX.—*Road.*—The Waltham Abbey and Cheshunt Councils are considering a scheme for the construction of a road to connect the two parishes. The estimated cost is £60,000.

FARNWORTH.—*Houses.*—The R.D.C. is negotiating for the purchase of 27 acres of land on which it is proposed to erect 250 houses.

FLAXTON.—*Bridges.*—A Committee has been appointed to inquire into a proposal to erect bridges at Towthorpe and Strensall.

GLOUCESTER.—*Road.*—The Corporation proposes to construct an arterial road from Stroud Road to Tredworth Road, at an estimated cost of £21,225.

GOSPORT.—*Houses.*—The Corporation has applied for sanction to borrow £61,000 for the erection of 122 houses on the Forton estate.

GRAYS.—*Houses.*—The U.D.C. is considering a scheme for the erection of 200 houses.

GUILDFORD.—*Houses.*—The T.C. is preparing a scheme for the erection of 700 houses.

HINCKLEY.—*Houses.*—The R.D.C. proposes to erect 150 houses in the district.

HOLYWELL. — *Embankment.* — The R.D.C. is inquiring into a scheme for the construction of an embankment over the River Dee.

JARROW.—*Houses.*—The T.C. proposes to erect 100 houses by direct labour.

KENT.—*Roads.*—A scheme is being prepared for the construction of by-pass roads at Sevenoaks, Tonbridge, Wrotham Heath, and Rochester.

LONDON (LEWISHAM).—*Houses.*—The B.C. are negotiating for the purchase of 43 acres of land for housing purposes.

LOUTH. — *Bridge.* — The R.D.C. proposes to erect a bridge over the canal.

LYE.—*Houses.*—The D.C. is considering a scheme for the construction of 100 houses at Lye and Woolescote.

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MEXBOROUGH.—*Houses.*—The Council has asked for sanction to erect 615 houses.

MORRISTON.—*Houses.*—The Swansea Corporation Housing Committee proposes to build 112 houses at Morriston.

MOTHERWELL.—*Bridge.*—A sum of £124,000 is to be spent on the construction of a bridge over the Clyde.

NEWBIGGIN.—*Footpath.*—The U.D.C. has instructed the Surveyor to prepare plans for the construction of a footpath round Newbiggin Bay.

NEWRY.—*Road.*—The U.D.C. proposes to construct a road from Water Street to Mill Street.

OLDBURY.—*Houses.*—The U.D.C. proposes to build 100 houses.

OVERSTRAND.—*Road.*—The C.C. Highways Committee has instructed the County Surveyor to prepare a scheme for the construction of a proposed road.

OVERSTRAND.—*Sea Defence Works.*—The Erpingham R.D.C. has applied for a loan of £2,500 for sea defence works.

PLYMOUTH.—*Reservoir.*—A sum of £135,000 is to be spent on increasing the capacity of the water storage reservoir by 50 per cent.

PRESCOT.—*Road.*—The U.D.C. has applied for a loan of £3,170 for the construction of a road from Warrington Road to Bretherton Road.

ROWLEY REGIS.—*Footbridge.*—The U.D.C. proposes to construct a footbridge over the Mouse Sweet Brook.

SALFORD.—*Houses.*—The C.B. has applied for a loan of £60,000 for the erection of 126 houses on the Littleton Road estate.

SCARBOROUGH.—*Sea Wall.*—The Corporation proposes to spend £14,181 on the construction of a sea wall and roadway at the base of the cliff at South Bay.

SHERINGHAM.—*Concrete Groyne.*—The U.D.C. is to spend £4,000 on the construction of a concrete groyne.

SUTTON.—*Road.*—A sum of £1,100 is to be spent on the construction of a road from Sherwood Park Road to St. James' Road.

SUTTON COLDFIELD.—*Houses.*—Application has been made for a loan of £50,000 for housing purposes.

TORQUAY.—*Quay.*—A sum of £20,000 is to be spent on the reconstruction of Beacon Quay.

TURRIFF.—*Reservoir.*—The Surveyor has been instructed to prepare plans for a reservoir.

WALTON.—*Sea Defences.*—The U.D.C. is considering a plan for the construction of sea defence works between Sloper's Nose and the Pavilion groyne.

WEYMOUTH.—*Road.*—A sum of £35,000 is to be spent on the construction of a road to the west of the breakwater.

WOLVERHAMPTON.—*Houses.*—The Council's housing scheme provides for the erection of 1,700 houses during the next two years.

WORTHING.—*Groynes.*—A sum of £450 is to be spent on the construction of groynes.

Tenders Accepted.

BENNINGTON.—*Bridge.*—The Holland C.C. has accepted the tender of Messrs. W. Wright & Son (Lincoln), Ltd., at £4,170, for the erection of a reinforced concrete bridge over the Hobhole Drain, at Sibsey.

CHICHESTER.—*Reservoir.*—The T.C. has accepted the tender of Messrs. Playfair, Toole & Co., Southampton, at £8,506 10*s.* 7*d.*, for the construction of a reinforced concrete reservoir.

CROYDON.—*Foundations.*—The C.B. has accepted the tender of Messrs. Peter Lind & Co., at £3,465, for the construction of concrete foundations.

DONNINGTON.—*Bridge.*—The Holland C.C. has accepted the tender of Mr. R. J. May, of Trowse, Norwich, at £5,279 6*s.* 9*d.*, for the erection of a reinforced concrete bridge over the South Forty-Foot Drain at Donnington.

GODSTONE.—*Concrete Houses.*—The R.D.C. has accepted the tender of Messrs. David Weston & Co., Ltd., for the construction of 24 concrete houses, roads, etc.

HUYTON-WITH-ROBY.—*Kerbs.*—The U.D.C. has accepted the tender of Mr. S. H. Bonsall, at 4*s.* 2*d.* per yard, for the supply of concrete kerbs.

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INVERNESS.—*Bridge.*—The C.C. has accepted the tender of Messrs. Sir Robert McAlpine & Sons for the demolition and reconstruction in reinforced concrete of the Findhorn Bridge, Tomatin.

ORSETT.—*Wall.*—The Essex E.C. has accepted the tender of Mr. W. Simms, of Stepney, at £1,875, for the construction of a reinforced concrete wall at Grays Arthur.

PERTHSHIRE.—*Concrete Bridge.*—The C.C. has accepted the tender of the Gray's Ferro-Concrete Co., Ltd., at £8,310, for the construction of a reinforced concrete bridge over the Forth at Drip.

SKEGBY.—The R.D.C has accepted the tender of Messrs. Fletcher & Co., Blidworth Colliery, at £585, for bridging the Soulkholme Ford in reinforced concrete.

WILLESDEN.—*Concrete Houses.*— The U.D.C. has accepted the tenders of Messrs. Thomas & Edge and Mr. E. M. Roberts for the erection of a pair of concrete cottages for demonstration purposes.

WILLESDEN.—*Concrete Houses.*— The U.D.C. has accepted the tender of the Calway Construction Co. for the erection of 100 concrete houses on the Brentfield estate.

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AERATED CONCRETE CO., LTD. (208507). Registered September 21. 97-8, Abbey House, Victoria-street, S.W.1. Manufacturers of cement, lime, etc. Directors: H. Le Marchant, K. P. Billner and C. Nicol.

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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EDITORIAL NOTES.

Porous Concrete.

APART from the development in the uses to which concrete may be put due to the better quality of cement and consequently stronger and more reliable concrete that is produced, other new uses are arising as a result of ingenuity and invention. One of these that has lately received prominence is the use of concrete as an insulating material. Under ordinary circumstances the aim of concreters is to produce the densest possible concrete by grading the aggregate so that the proportion of air-voids is at the minimum, but if the reverse object be aimed at, viz. the maximum of air-voids, then the concrete becomes a material of high heat-insulating value. For obstructing the passage of heat, a vacuum is the best medium, as witness the vacuum flask, and the next best is air; hence the more air-voids a concrete contains the better will it insulate or prevent the passage of heat.

One means of producing air-voids in concrete is to mix cement with hydrated

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lime and aluminium powder; when water is added to this mixture a chemical action takes place liberating hydrogen gas which causes the cement to swell and become filled with bubbles. When the cement hardens a honeycombed mass is obtained which has a quarter the weight of ordinary concrete, and which also has good insulating properties. It will be obvious that such a concrete is correspondingly weaker than ordinary concrete, but it is claimed to be sufficiently strong for partitions and walls of small houses. For housing purposes a porous concrete would provide a cool dwelling in summer and would prevent condensation in winter.

Another method of producing a porous concrete is to use only coarse material as aggregate, eliminating the sand that is employed for filling the voids in an ordinary concrete. When the coarse aggregate is itself porous or honeycombed, such as clinker, the concrete becomes still more porous and heat insulating. The same principle of porosity has lately been applied to the construction of tennis courts with the object of providing a court which would be highly absorbent and therefore fit for play immediately after the heaviest rain shower.

In all the attempts to adapt a porous concrete to practical uses, it must be realised that porosity is only obtained by the sacrifice of strength, and it is only when a high degree of strength is not essential that porous concrete can be employed.

Jointless Floors.

WHEN caustic magnesia is mixed into a paste with a solution of magnesium chloride a cement is formed which sets in a few hours to a hard marble-like mass, well adapted for the surfacing of floors. Hitherto such floors have generally been laid by specialist firms giving the material various trade names, but the Building Research Board of the Department of Scientific and Industrial Research has rendered a public service in publishing a report (Building Research Bulletin No. 1, H.M. Stationery Office, 1s. net), giving full details of the composition of the cement and the method of use, so that it should be possible in future for the ingredients of magnesium oxychloride cement—to give it its proper title—to be purchased in a competitive market and for the cement itself to be used by any builder accustomed to the manipulation of concrete.

The cement is mixed with sawdust, cork, sand, asbestos, etc., as fillers or aggregates, and being white it can be readily coloured by incorporating the usual mineral pigments. Floors laid with magnesium oxychloride cement have a smooth surface giving them a better appearance than an ordinary concrete floor, and they are particularly suitable for public buildings where a sanitary and non-inflammable floor is desirable. Such floors cost appreciably more than a concrete floor, and they can only be used internally, because the cement loses its strength and finally disintegrates under the action of water.

A serious defect of magnesium oxychloride cement is its tendency to corrode iron and steel with which it comes in contact, but under the auspices of the Building Research Board some useful research has been undertaken which has led to the discovery that by substitution of the magnesium chloride constituent of the cement by ferrous chloride the corrosive tendency is eliminated. The only disadvantage of the modified cement is that it has a brown colour when set and does not lend itself to colouring by pigments.

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A prefatory note to this Bulletin states that it is the first of a series embodying the results of investigations carried out at the Building Research Station. If future bulletins are as clearly written and contain information of such direct practical application as this they should be of great service to all engaged in building and constructional work.

Coloured Concrete.

An announcement by Mr. G. Topham Forrest, F.R.I.B.A., Architect to the London County Council, that he proposes to use coloured Portland cement on the new L.C.C. housing estates, will be received with interest by those concerned with the artistic possibilities of concrete. Although much may be done to avoid a monotonous effect by variations in design and lay-out, the possibilities of coloured effects made practicable by the use of cement washes and coloured cement should be of great value to the architect. Use has already been made of colour in concrete-block houses and in the monolithic concrete houses at Welwyn ; in the former case by facing some of the blocks with a layer of concrete coloured with oxides or pigments and using the coloured blocks to form string courses, and in the latter case by covering the walls with a wash of cement coloured pink, blue, buff, and other shades. In this way we shall see houses reminiscent of the old colour-washed cottages which are such a picturesque feature of many parts of the country.

"The Concrete Year Book."

We may remind our readers that the 1926 Edition of "The Concrete Year Book," edited by Dr. Oscar Faber, O.B.E., D.Sc., M.Inst.C.E., and H. L. Childe, will be ready on January 1 next. This edition, extended to over 400 pages, has been thoroughly revised, and a great deal of new matter of every-day use to all connected with the concrete industry has been added. Those who are not familiar with this volume may gather some idea of its value by the fact that an engineer told us the other day, "I use your 'Year Book' more than any other work of reference" ; while those who already know the book and wish to make sure of securing a copy are advised to place an order early in order to avoid disappointment, as in spite of large printing orders previous editions have always been exhausted well before the end of the year. In addition to the mass of useful data in the Handbook Section, the volume contains the only complete Directory of every branch of the concrete industry and a Catalogue Section of over 200 pages giving full particulars of all the chief items of machinery, plant, products and materials used in the industry. Although the volume has further increased in size the price remains the same at 2*s.* 6*d.*, and an order form is given on p. xx.

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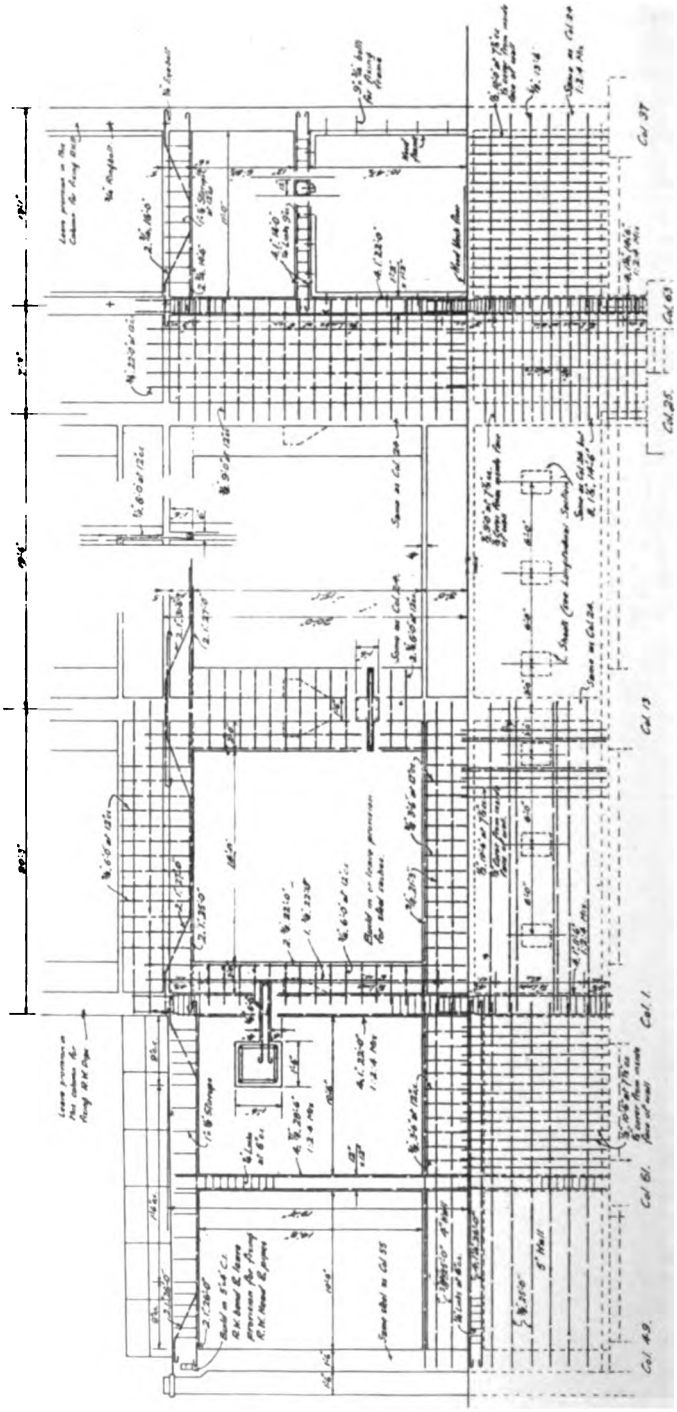

NORTH ELEVATION.

BASEMENT, GROUND, AND FIRST FLOORS.

Reinforced Concrete Factory in South-West London. (*See p.* 657).

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Reinforced Concrete Factory in South-West London.

THIS factory, recently completed at South-West London, is a good example of the eminent suitability of reinforced concrete for this class of building, where maximum light and floor space are required on a restricted site. The factory is almost surrounded by other tall buildings, which renders it difficult to obtain photographs of the whole structure.

The main block is 60 ft. wide by 166 ft. long, and consists of basement, ground floor, and four floors over, with the usual flat roof. The building is stepped back 20 ft. at the fourth floor in order to avoid obstructing the light of the adjacent property. On the east side the factory is extended at first-floor level, providing an additional space 166 ft. by 24 ft.

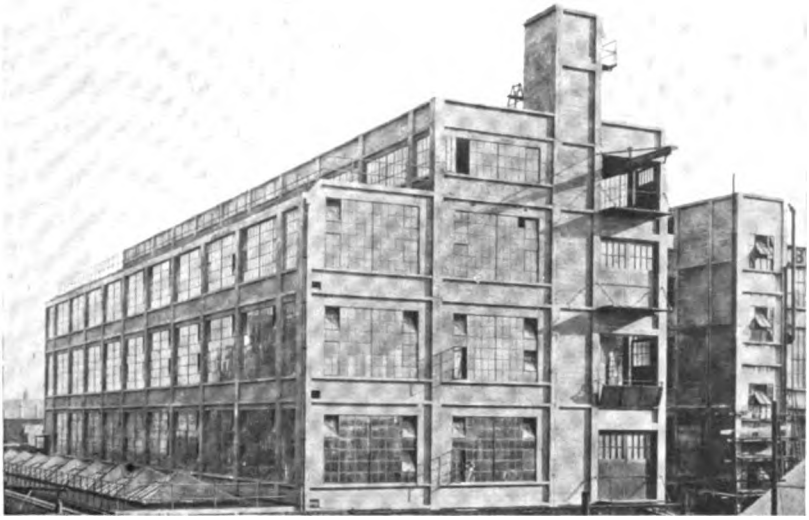
The ground story is 20 ft. high, and is equipped with overhead travelling cranes in each bay, the rails being carried on column brackets. First, second, and third floors are 14 ft. high and the fourth floor 9 ft. 6 in. The basement, which will be used for storage purposes, is 8 ft. 9 in. high, and is enclosed by a retaining wall 5 in. thick. The total height of the building from basement to main roof is 81 ft., and to the top of the lift towers 100 ft.

The floor loads generally are 2 and 2½ cwts. per sq. ft., but over certain areas are as high as 5 cwts. per sq. ft. Floor slabs are 5 in. thick, with 12 in. by 7 in. secondary beams at 6 ft. 6 in. centres and 18 in. by 9 in. main beams at 16 ft. 6 in. centres. The roof slab is designed for 1 cwt. per sq. ft. load, and is 4 in. thick, the secondary and main beams being 10 in. by 5 in. and 14 in. by 7 in. respectively.

The internal columns are 26 in. square in the basement and 12 in. square under the roof. The side columns are 18 in. wide throughout their height, the depth varying from 30 in. to 12 in.

The foundations are taken down to compact ballast, and the loads, which in some cases exceed 300 tons per column, are distributed to about 4 tons per sq. ft.

A 7-ton travelling crane is fitted on the third floor and arranged to pass a few feet beyond the west side of the building to enable heavy goods to be unloaded from lorries in the yard below. The



Reinforced Concrete Factory in South-West London.

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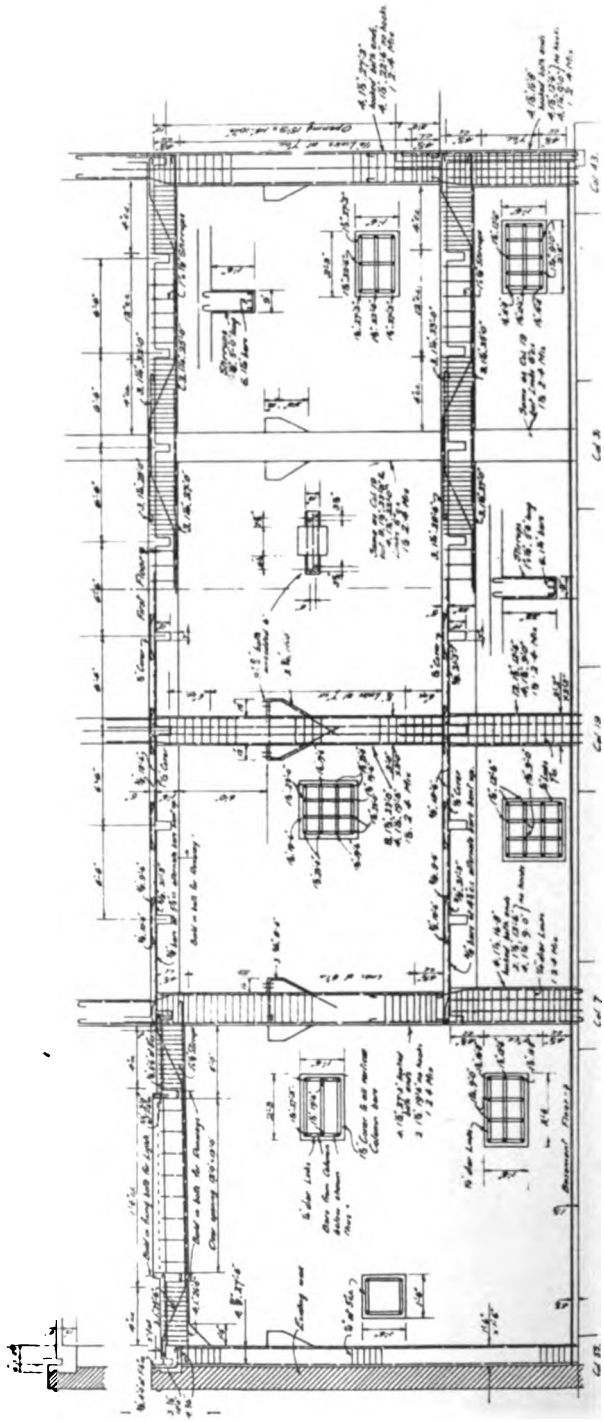

BASEMENT, GROUND, AND FIRST FLOORS.

Reinforced Concrete Factory in South-West London.

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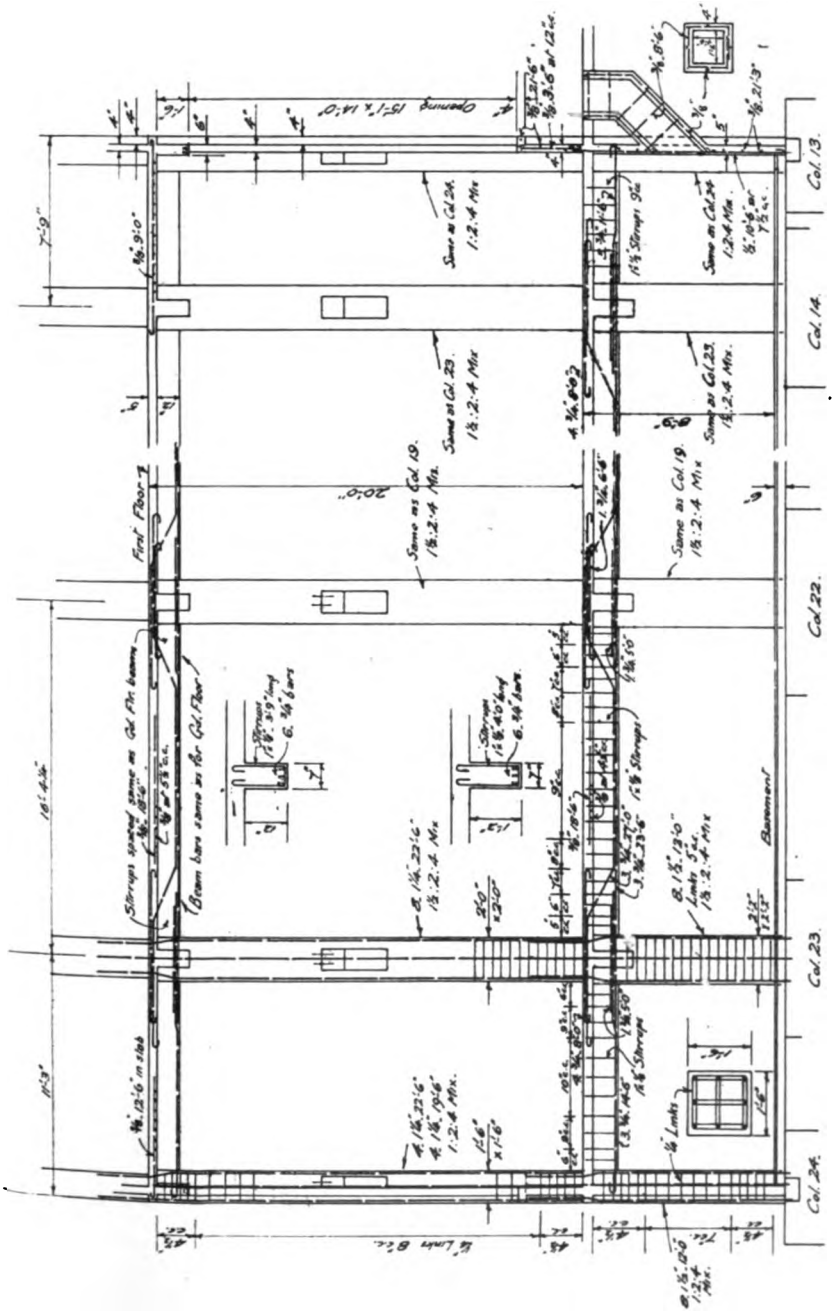

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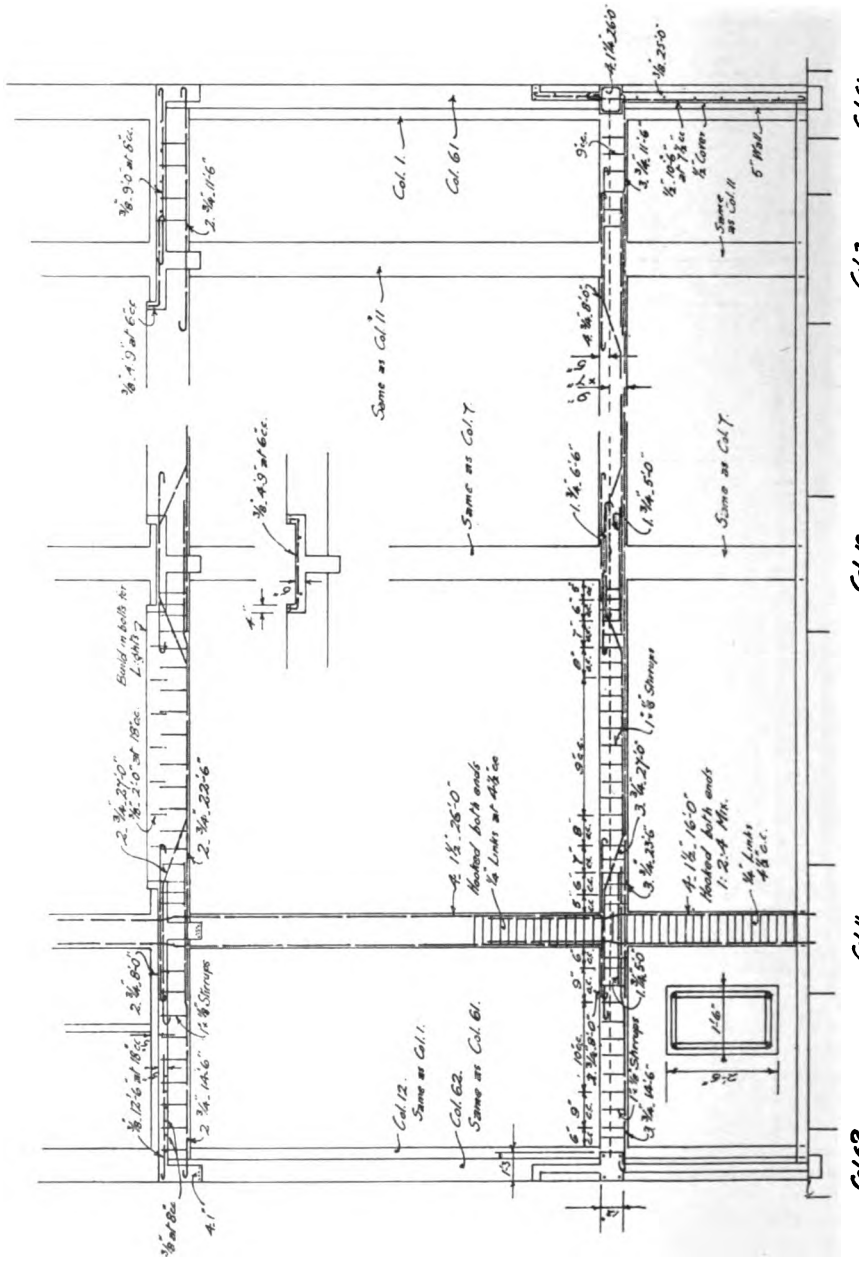

PART EAST ELEVATION: BASEMENT, GROUND, AND FIRST FLOORS.

Reinforced Concrete Factory in South-West London.

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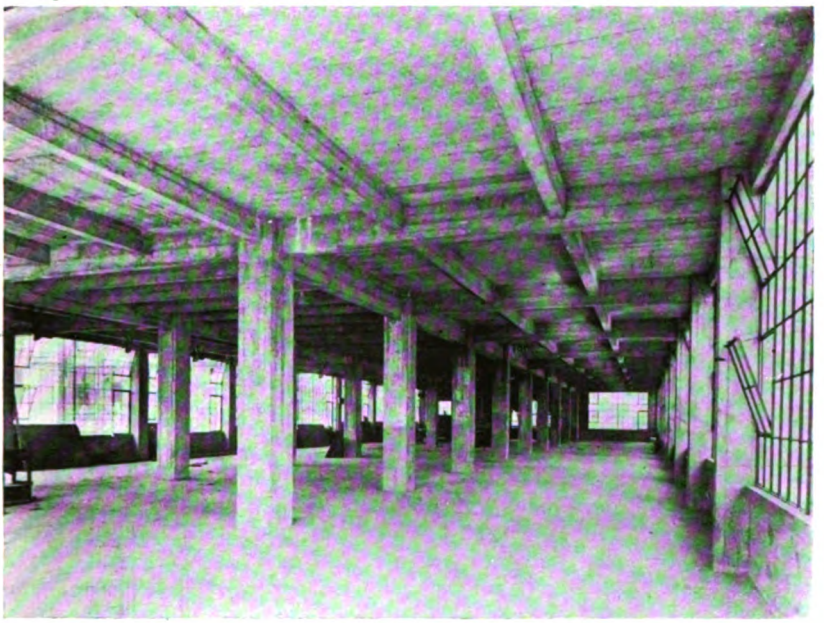

Interior View.

fourth floor has a light runway which projects from one end of the factory.

Two electric lifts of 2-tons capacity are provided, one at each end. The staircase is totally enclosed by concrete walls and fireproof doors, and additional means of escape in case of fire is provided from each floor by concrete bridges leading to a similar staircase in an adjoining building.

Rapid-hardening Ferrocrete cement was used, and in conjunction with Ham River shingle and sand resulted in a highly satisfactory concrete. Advantage was taken of the quick-hardening cement, the formwork being removed much earlier than is usually possible, with a consequent saving in the amount of timber used and considerable speeding up of the work.

The factory was designed by Messrs. Lewis Rugg & Co., of Westminster, and the contractors were Messrs. John Mowlem & Co., Ltd.

Form Panels for Concrete Walls Bolted to Brick Wall.

In the erection of an ice storage building at Long Island an 8-in. concrete wall was built against an existing brick wall, and a novel method of holding the formwork in position was adopted by Mr. Henry Wilcox, the contractor. The brick wall was about 36 ft. long by 20 ft. high, and as the new building was 49 ft. wide by 40 ft. high in the clear it would have been difficult to brace the forms. Moreover, the concrete was carried up in 14-ft. lifts, and the pressure would probably have burst the brick wall. Accordingly it was decided to bolt the form panels to the brick with expansion bolts. A rough test of the holding power of a $\frac{5}{8}$-in. shield was made by screwing in the bolt till the brick spalled. This test indicated that holes 2 in. deep would give a large margin of safety. So a standard form clamp spacing was laid out on the brick wall, one end of the clamp rods threaded and the form put up. It held perfectly. The cost per hole for laying out, drilling and shield was about 2*s.* 6*d.*, or 3*d.* per sq. ft. of wall. This compared favourably with bracing, even for ordinary story heights. Moreover, the method practically eliminated unbalanced stress in the existing wall.

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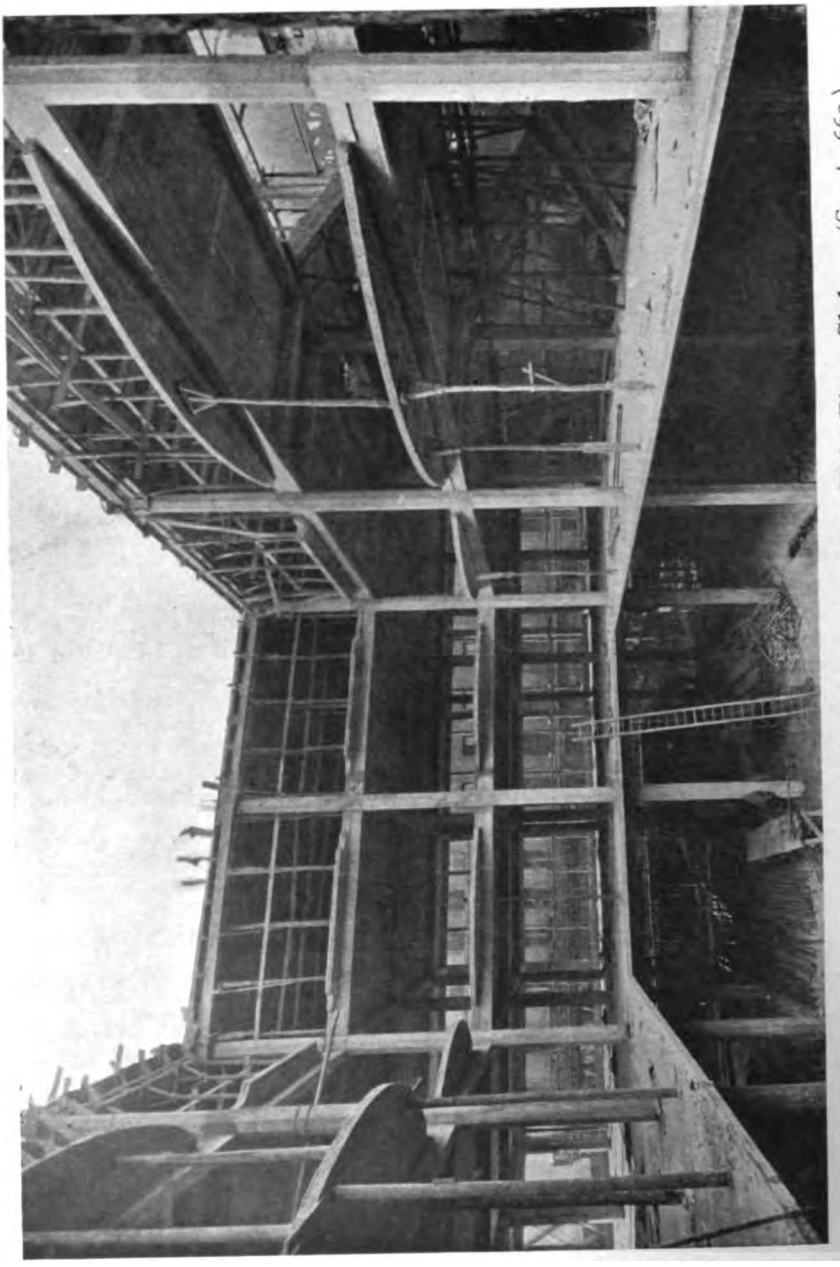

Reinforced Concrete Building in Paris: Showing Thin Floor Slabs. (*See p.* 663.)

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Novel Type of Thin Floor Slab.

By Paul Calfas.

THE new store in the Rue de Rivoli, Paris, illustrated herewith, is remarkable for the design of the floor slabs, used here for the first time by the engineer, M. Gellusseau. The building is on a corner site, with frontages of 196 ft. and 111½ ft., and as will be seen from the photographs the upper floors surround a large central hall running from the ground floor to the top of the building. The building is of reinforced concrete frame construction, with stone frontages.

The floor slabs are 19 ft. 8 in. by 21 ft. 8 in., and are designed for a live load of 72 lbs. per sq. ft. The thickness of these slabs is 5½ in. at the edges and 3½ in. at the centres, the difference in thickness being obtained by forming a flat top and a depressed under-surface, as shown in the section on p. 664. Undoubtedly this is the thinnest floor ever built with a span of 21 ft. 8 in. The design of the floor is based on the reduction of shear stress, owing to the arch-shape of the slabs, and also on the difference in bending moments at the centre.

The formula used for the design of

Reinforced Concrete Store Building in Paris.

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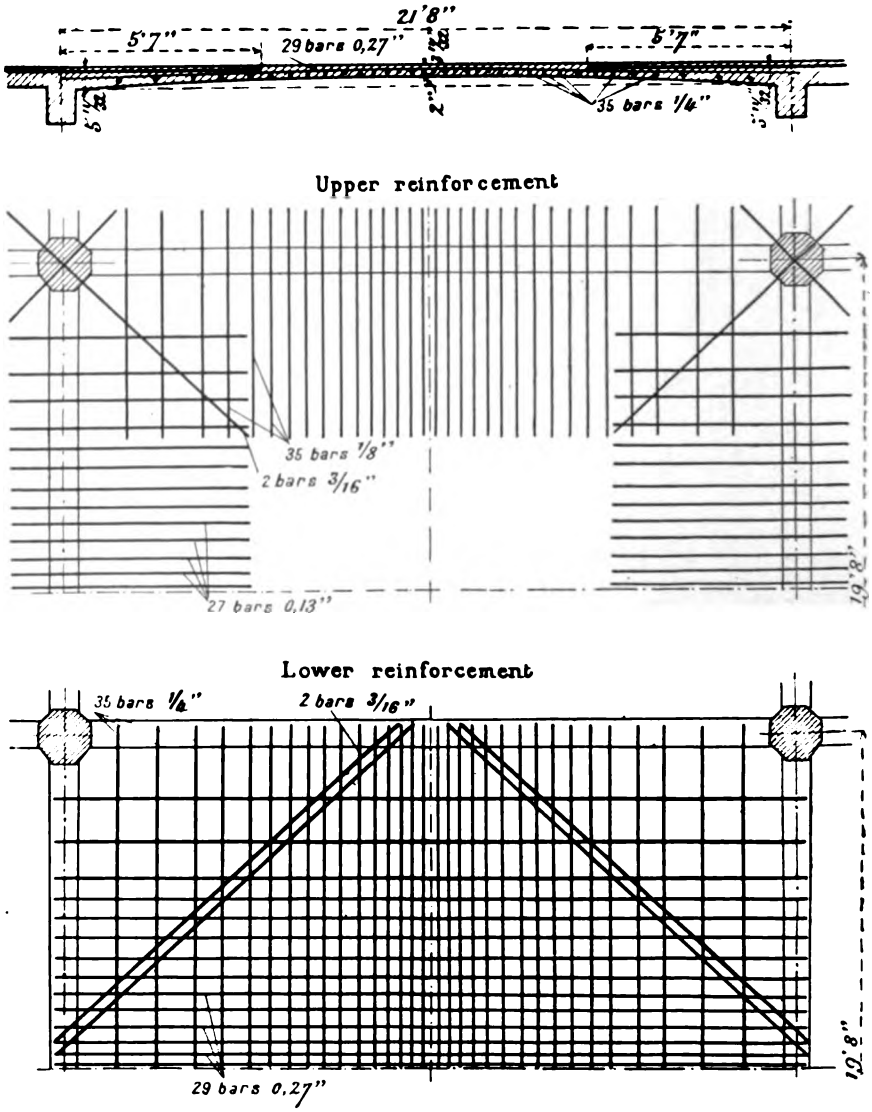

Reinforced Concrete Building in Paris: Design of Floor Slabs.

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these slabs (21 ft. 8 in. by 19 ft. 8 in.) is as follows :—

A. When B.M. is calculated for a 6m. 00 span)—

(*a*) At the springs—

(1) Dead load, B.M. $= \frac{W^2}{18 \cdot 5}$

(2) Live load, B.M. $= \frac{W^2}{22 \cdot 5}$

(*b*) At the centre—

(1) Dead load, B.M. $= \frac{W^2}{67 \cdot 7}$

(2) Live load, B.M. $= \frac{W^2}{40 \cdot 6}$

B. (When calculated for a span of 6m. 60)—

(*a*) At the springs—

(1) For dead loads, B.M. $= \frac{WL^2}{22 \cdot 3}$

(2) For live loads, B.M. $= \frac{WL^2}{27 \cdot 2}$

(*b*) At the centre—

(1) For dead loads, B.M. $= \frac{WL^2}{81 \cdot 5}$

(2) For live loads, B.M. $= \frac{WL^2}{49}$

From these various values, the dimensions and reinforcement of the floor slabs as shown in the drawings have been arrived at. A high-resisting cement is employed for the concrete mixture.

The use of rapid-hardening Portland cement, together with the use of formwork of standard dimensions, permitted a very rapid execution of the work. The first part of the building, representing 17,225 sq. ft. in surface, was completely carried out in less than eight months, and the framework of the second part, which covers an area of 5,400 sq. ft., was completed within three months. The architect is M. Perret.

Correspondence.

Concreting in Cold Weather.

SIR,—We have read an interesting article in your November issue entitled "Concreting in Cold Weather," and do not doubt but that this article will be of great service to contractors and others using concrete should we experience a hard winter.

At the same time we feel that it will probably interest you and your readers to know that if concrete be made with an aluminous cement, such as "Ciment Fondu," manufactured by the Lafarge Aluminous Cement Company, Ltd., or (I understand) the "Lightning" brand of the Cement Marketing Company, Ltd., no precautions whatsoever need be taken in cold weather, as the chemical action is so rapid that heat is developed in the concrete itself.

The advantages of this fact are immediately obvious: protection involves considerable expense and always entails a certain amount of risk, so that the majority of engineers insist on all concrete work being stopped when the temperature is below freezing point. There is absolutely no need whatsoever to stop concreting work during the hardest frosts experienced in this country, as concrete made with aluminous cement remains quite unaffected under such conditions and protection is superfluous.

LAFARGE ALUMINOUS CEMENT CO., LTD.,
JOHN G. KAY,
Managing Director.

[*** While we agree that aluminous cement is more immune than other cements from attack by frost, we consider that it would be unwise to accept too literally our correspondent's recommendation that "no precautions whatsoever need be taken in cold weather" when using aluminous cement. This cement sometimes has an initial set of 2 or 3 hours during which period no heat is evolved, and in a hard frost it may be possible for an unprotected concrete to become frozen before setting commences. Further, it is possible for the heat developed during the setting of aluminous cement to be dissipated almost immediately—for example in a thin layer of plaster—and in such cases the exterior surface might suffer in a hard frost. Our view is that aluminous cement concrete or mortar laid in frosty weather should be covered up to protect the surface and conserve the heat.—ED.]

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Colouring a Concrete Bridge.

THE scheme for the widening and maining of the Dalemain Road, a link between Penrith and Ullswater Lake, 3½ miles in length, was commenced by the Cumberland County Council at the beginning of this year, the estimated cost of the work being £23,712. The road passes through Dalemain Park, situated in which is a narrow County bridge possessing very awkward road approaches. The widening scheme provides for a road diversion and new bridge in the Park, and in order that the new bridge should harmonise with the house, which is constructed of pink sandstone, it was considered the new bridge should possess a similar appearance.

[*Mr. G. O. Lockwood, A.M.Inst.C.E., F.S.I., Cumberland County Surveyor.*

Dalemain Bridge, Cumberland.

The new bridge consists of a single span of 34 ft., and is 30 ft. between parapets. The arch has a rise of 4 ft., being 2 ft. 3 in. lower than the existing arch, but in order to provide the same area of waterway the span was increased 5 ft. The new structure is designed on the Hennebique system with mass concrete abutments, reinforced concrete arch, spandril and wing walls. The under side of the arch is the natural cement colour, but the reinforced concrete spandrils, wing walls, and parapets contain a percentage of colouring so that a pink sandstone effect is obtained. In order to obtain the correct colour a sample of the pink sandstone was first obtained, and after a careful reconnaisance of the district it was decided to use pink Shap granite and Bowscar red sand for the concrete aggregate.

The Reinforced Concrete Construction Company, of Manchester, the contractors, carried out the experiments for the colouring of the concrete in order to ascertain the correct percentage of red oxide, yellow ochre and black that would have to be added to the cement in order to obtain the desired colour. The colour decided upon contained 1½ per cent. of colouring matter, and cement tests were next carried out with a view to ascertaining whether the colouring matter affected the strength of the cement. These tests showed an average tensile strength of 655 lbs. per sq. in. after 7 days. Messrs. G. & T. Earle, Ltd., were then entrusted with the mixing of the correct percentage of colouring with the cement, so that the coloured cement could be delivered on the site in bulk. The proportions of aggregate, sand, and cement were very carefully adhered to, and the result is an even coloured concrete throughout the thickness of the various members forming the bridge, the strength of which, as proved by the experiments made beforehand, has been in no way impaired.

The cost of constructing the bridge amounts to £1,776. The complete scheme of widening, which is nearing completion, is being carried out under the supervision of Mr. G. O. Lockwood, A.M.Inst.C.E., F.S.I., County Surveyor and Bridgemaster.

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Reconstruction of Pandy Bridge, Monmouth.

By W. L. Scott, A.M.Inst.C.E.

THE new Pandy Bridge at Cross Keys, Monmouth, which is 70 ft. clear span and 36 ft. wide between parapets, replaces an older bridge consisting of two steel girders forming the parapets about 25 ft. apart, carrying a roadway on cross girders and timber joists. In view of the difference in size of the old and new bridges it was necessary completely to demolish the former structure and replace it by the new bridge, which is a single-span arched bridge in reinforced concrete supported on new abutments composed of mass concrete.

The general arrangement of the new structure is as follows :—The four main arches at 9 ft. centres in the transverse direction thrust against a sleeper beam of reinforced concrete, which in turn transfers the pressure to continuous abutments formed in mass concrete. These arch ribs are rectangular in form and are 21 in. deep at the crown of the arch, increasing to 30 in. deep at the springings, and have a constant width of 30 ins. The reinforced concrete roadway slab is carried for the centre portion of the bridge directly on the arched ribs and on each side on light flexible columns connecting the deck platform to the arch ribs. No transverse beams have been introduced in the deck of the bridge, the main reinforcement in the roadway slab, which is 9 in. thick, being placed in the transverse direction.

The deck slab is cantilevered on each side of the main ribs and carries the footpaths and parapets, the requisite accommodation for the pipe mains being obtained in the space provided between the surfacing to the footpaths and the deck slab in the cantilever supporting it. This space is earth filled and forms a simple method of accommodating the necessary pipes and cables, and also permits of repairs being made to them in the usual manner without interfering in any way with the reinforced concrete structure and without greatly interfering with the traffic passing over the bridge.

On each side of the bridge, in line with the parapets, spandril walls of reinforced concrete have been provided and these are supported by internal counterforts which connect with the mass concrete. A front curtain wall along the river face of each of the abutments is also provided and stiffened by counterforts. The space enclosed by these spandril and abutment walls is filled solid up to the roadway level, the arrangement being made quite clear in *Fig.* 2, which gives the general plan of the bridge.

A considerable volume of water passes under this bridge during the winter months, and it was this fact that made it desirable to have an arch bridge of the open spandril type. With this arrangement, when the river is in flood the water can pass freely through the spandril of the bridge with the minimum obstruction therefrom.

This bridge was designed to carry two trains of the Ministry of Transport standard loading, comprising an engine of 20 tons followed by three trailers of 13 tons each. The specified loading of these vehicles given above includes for impact stresses of 50 per cent., which is therefore presumed to affect the whole of the structure subjected to this loading.

The rise of the main arch ribs is 11 ft. and the rise span ratio is therefore approximately 1/6·4. The longitudinal arch axis has for its form a parabola of the second degree, this being also the form of the pressure curve upon the assumption that the loading is uniformly distributed.

It will be seen from the longitudinal section (*Fig.* 3) that the deck platform is entirely separated from the abutment retaining walls, and that the whole load coming on the roadway and footpaths over the bridge proper is transmitted to the supports directly from the arch ribs. This condition is essential if the assumptions customary in the design of such an arch bridge are to be realised in fact.

Whilst the hingeless arch from the architectural and other points of view is a very satisfactory solution for many bridges, there are several difficulties which arise in the design of this type, and careful consideration requires to be given to these problems. These diffi-

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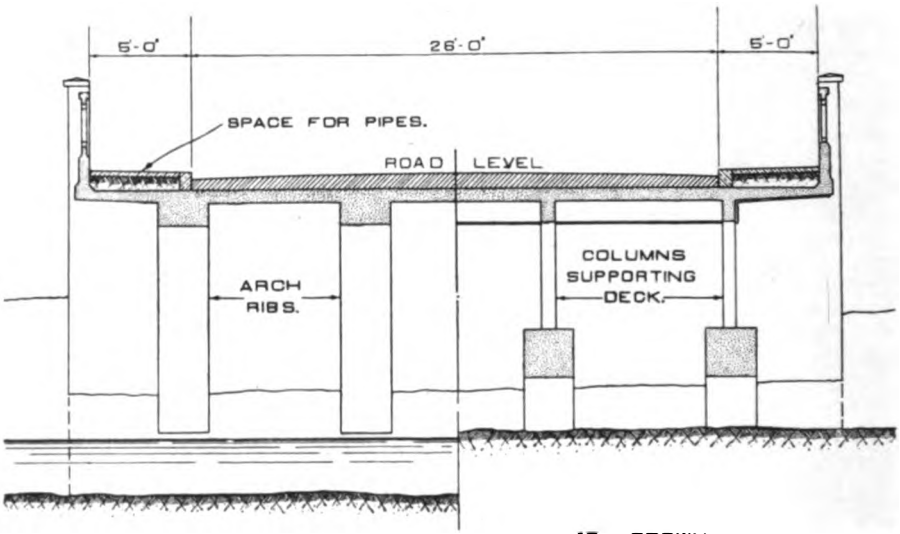
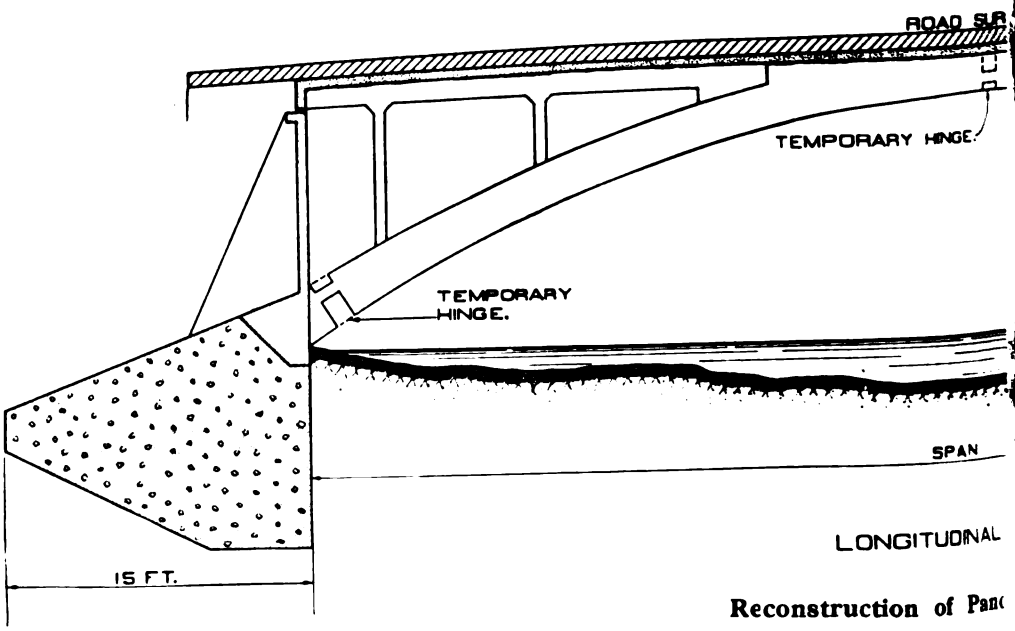


AT SPRINGING. AT CROWN.

CROSS SECTION.



LONGITUDINAL

Reconstruction of Pan

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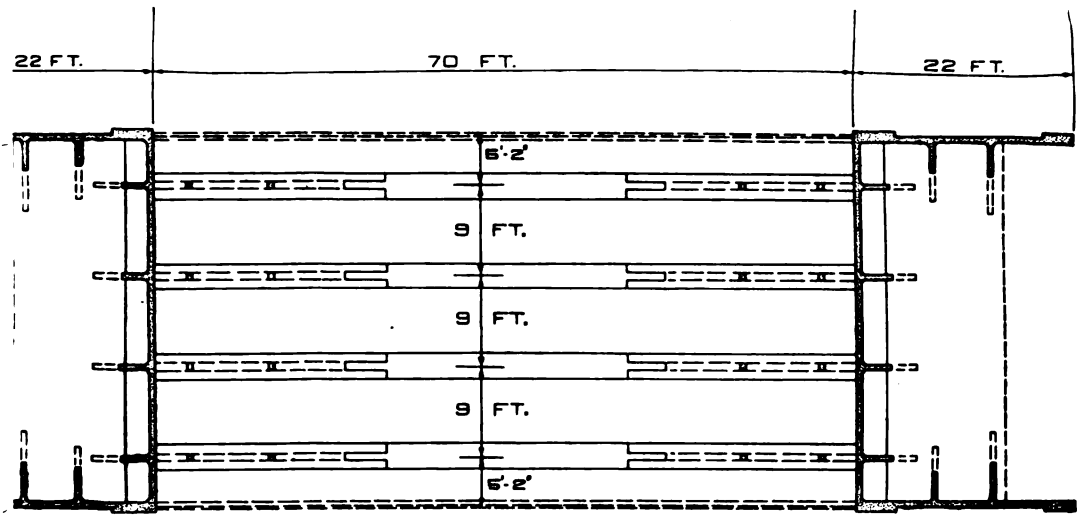
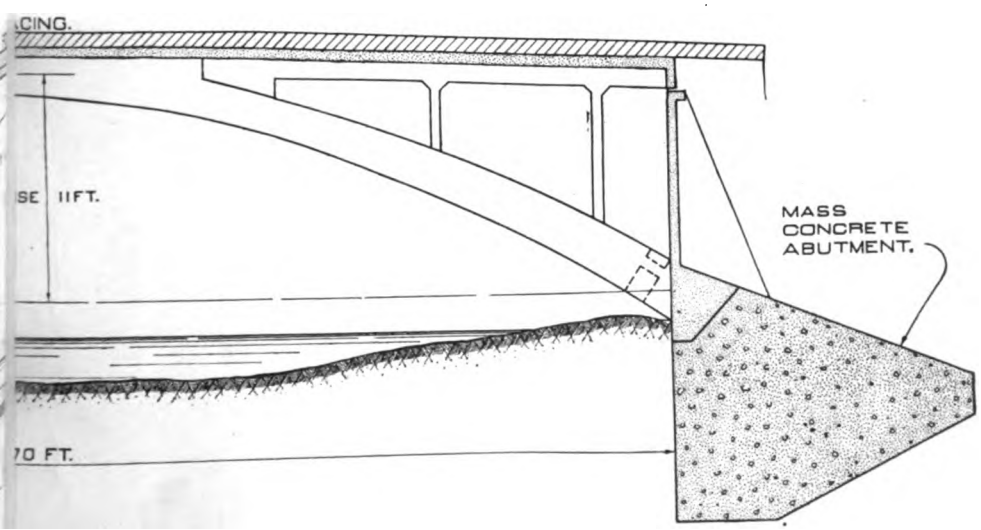


PLAN.



SECTION.

Bridge, Monmouth.

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culties may be referred to as the secondary stresses taking place in arches of reinforced concrete, and are sometimes of greater importance than the stresses directly resultant from the loading. The shrinkage of the concrete, however carefully the work may be constructed with a view to diminishing this, produces moments and stresses in the arch ribs. Also, the elastic compression of the arch ribs resultant from their being subjected to the weight of the bridge and superimposed loading causes further deformation, and produces secondary moments and stresses of considerable relative magnitude. Furthermore, unless the bridge abutments are founded upon material reasonably incompressible, such as rock, the abutments must inevitably settle slightly when the load is brought upon them, and this again, although not causing a shortening of the arch axis, produces deformation having the same effect, i.e. flattening of the bridge at the crown. Finally, a drop in temperature causes the concrete to contract, and this adds further stresses of the same sense as produced by the above cumulative effects.

The total flattening of the arch resultant from these deformations, if not neutralised in some way or other, produces very serious stresses, and the greater proportion, which is resultant from linear variations in the arch rib, causes stresses which are irrespective of any amount of reinforcement that may be provided in the rib for taking care of them, since these latter stresses are the result of a definite strain taking place after the ribs are completed and fixed to the abutments. In any bridge having a span similar or greater than this bridge the dead weight of the structure comprises the greater proportion of loading to be dealt with.

Having regard to this, and to the fact that most of the shrinkage taking place in the structure occurs before the work is entirely completed, it will be seen that the introduction of temporary or provisional hinges eliminates the stresses that would be produced during the construction of the work. For this reason temporary hinges were employed in this bridge, three of them being placed in each rib (two at the springings and one at the crown), enabling the statical solution for three-hinge arches to be employed in calculating the stresses produced by the dead weight of the structure.

The serious secondary stresses produced by the flattening of the arch due

Reconstruction of Pandy Bridge, Monmouth.

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to the above effects have been studied for many years by engineers specialising in reinforced concrete, and particularly by Considère, who evolved temporary hinges as a means of overcoming this difficulty. A temporary hinge consists essentially in local reduction in the cross-sectional area. This local reduction requires to be made in a special manner, and the reduced section must be specially reinforced in order to make it capable of carrying the high compressive stress coming upon it during the construction of the work.

Although by the provision of temporary hinges during construction all the secondary stresses due to the weight of the bridge can be eliminated, there remain still the secondary stresses resultant from the superimposed loading and the stresses produced by the proportional shrinkage of the concrete taking place after the hinges have been concreted in and the work completed. The deformation producing these stresses when brought upon the arch ribs is permanently present throughout the life of the structure, and causes unequal distribution of maximum stresses on the intrados and extrados of the arched ribs.

By placing the temporary hinges some distance above or below the arch axis moments can be introduced into the arch ribs to any extent required, and by causing the moments resultant from the eccentricity of the temporary hinges to be of opposite sense and equal to those produced by the amount of equivalent arch shortening taking place after the completion of the work the whole of these secondary stresses can be eliminated.

Fig. 3, which is a longitudinal section of the bridge, shows the arrangement and position of the hinges, and also shows them in their eccentric positions for the reason described above.

In reinforced concrete work of this class the quality of the concrete is of the utmost importance and for this bridge it was specified to be 1 : 1½ : 3, the exact proportions of the larger and smaller aggregate required to produce a properly dense concrete of course being decided by test.

The maximum stress assumed on the steel reinforcement was 16,000 lbs. per sq. in., and the maximum permissible stress on the concrete 750 lbs. per sq. in. This stress, of course, was considerably increased in the case of the spiralled temporary hinges, such higher stress being necessary in order that these may properly function.

The bridge was designed by Messrs. Considère Constructions, Ltd., to the general plans of Mr. S. A. Bennett, A.M.Inst.C.E., F.S.I., County Surveyor of Monmouthshire. The work has been carried out in a very satisfactory manner by Messrs. Barnes, Chaplin & Co., of Cardiff.

Effect of Tornado on Buildings.

THE effects of the tornado which resulted in the devastation of large districts in Illinois last March and over 800 deaths, have been investigated by a committee of the Western Society of Engineers. The tornado ranged in width from a quarter of a mile to one mile, and travelled at a uniform speed of about 60 miles per hour for a total distance of 218 miles. The report of the committee is fully illustrated with photographs of ruined buildings and devastated towns, in some of which not a structure is still standing. The only buildings left intact, or practically intact, are those of reinforced concrete construction, and in its conclusions the committee emphasises the advisability of using reinforced concrete wherever practicable in the work of reconstruction.

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Concrete Cottage Construction.

In his presidential address at the inaugural meeting of the 1925–6 session of the Institution of Structural Engineers last month, Sir Charles Ruthen, Director-General of Housing, dealt at some length with the housing problem. After setting out the essential requirements of a dwelling-house, he said a number of concrete methods complied almost entirely with all of them, such as weather-resistance, stability, non-conductivity of heat and cold, cleanliness, rapidity of erection, and the employment of a minimum of skilled building labour.

During the past twelve months or so, he continued, no less than 20,000 houses of various new and alternative methods of construction had been erected in various parts of the country, and the experience gained as a result had been exceedingly valuable.

The question of condensation was one of the most difficult to be satisfactorily overcome, and it was perhaps well to state that in the main condensation depended more upon the surface of the wall than upon its composition. A polished surface was slow to absorb heat from the air and remained cold, and consequently was liable to cause condensation. If, in addition, surfaces were painted or coated with cement so that they could not absorb moisture, then the condensation was more noticeable. Hard plasters were not suitable for concrete work, and the coating of the interior walls of a house with paint or cement-wash was unwise. Theoretically, an absorbent surface of a wall did not prevent condensation, but by the absorption of the moisture prevented condensation from showing.

Generally speaking, reinforcement was unnecessary in cottage building, excepting for the purpose of strengthening certain pre-cast parts or supporting shuttering in concrete *in situ* work or for giving alignment when laying blocks. If the strength of the structure depended upon the reinforcement, that must be properly protected. In slabs of less than 1½ in. thick it was difficult to lay reinforcement so that it was properly protected.

Complicated beds in the use of concrete blocks were unwise. They often led to unequal pressure, and all the evils that resulted therefrom. In narrow-width blocks complicated joints were worse than useless, bedding was difficult, and grouting more so.

In concrete block work the simpler the block the more satisfactory from all points of view. If the concrete block were of such size and shape that it could be bonded with the ordinary brick it was advisable to do so, for there were many parts of cottage building where brickwork was most satisfactory. The size of the block should be such that the bricklayer or mason could handle it with the left hand, without laying down his trowel.

New methods of construction need not of necessity be ugly in execution. Artistic houses were possible by the use of other materials than brick. Quite apart from the entire construction of houses in concrete, undoubtedly there were parts of all houses and other buildings which were cheaper and quicker to construct in concrete in various forms, and there were other parts which were cheaper and quicker constructed of bricks. Some parts might be equally satisfactory constructed of either material. A unity of the two materials, if properly effected, would result in considerable saving of labour, materials, and cost, and an added speed in erection as well as an increased national output. The more general use of concrete below the ground level in nearly all buildings, large and small, would effect a great saving in the highest skilled labour and in the most valuable cottage building material.

There was no doubt that concrete properly used was the most reliable substitute for bricks in permanent cottage building, although many of the concrete houses erected during the past few years were not so satisfactory as could be desired. That was not the fault of the material, but of its application, and the various difficulties in regard to the use of concrete for cottage building were being rapidly overcome. The number of houses erected throughout the country under various alternative methods of building indicated that a serious effort had been, and was still being, made to supplement the output of brick houses, and he felt confident that the experience gained would be of lasting benefit to national housing as well as to the ultimate benefit of all classes of building work, and all ranks of the building industry.

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Combined Bending and Direct Force.

A Practical Method for the Design and Analysis of Rectangular Sections.

By Charles E. Reynolds, B.Sc. (Eng.) Lond.

THE object of this article is to present a practical and comparatively simple method of designing and analysing reinforced concrete members of rectangular section that are subjected to bending moments and direct thrusts or pulls. The treatment will be limited to those cases where the relative magnitude of the bending moment and direct force is such that both tensile and compressive stresses are induced in the section under consideration.

Let N equal the direct force (tension or compression), and B equal the bending moment. It is usual to replace N and B by a single force N acting at an arm e where $e = \frac{B}{N}$. Strictly speaking, this arm, or eccentricity, should be measured about an axis passing through the centroid of the stressed area of the section, but in the majority of treatises on the subject it is considered that this eccentricity is measured about the centroid of concrete section. Both the "correct" and the usual method lead to involved formulæ that are only conveniently capable of solution by the use of curves or tables. The method here proposed differs, however, from these, since the eccentricity is measured about an axis that is equidistant between the lines of action of the resultant tension and compression induced in the section, and it is claimed that for large values of e the accuracy of the results obtained by its application compares favourably with those obtained by the usual method, and are more readily obtained.

BENDING AND TENSION.

When N is a tensile force, the forces acting on a section are as shown in *Fig.* 1 where

a = lever arm or total distance between the lines of action of T and C.
T = resultant tension.
C = resultant compression.

Taking moments about the line of action of T:—

$$Ca = N\left(e - \frac{a}{2}\right)$$

$$C = \frac{Ne}{a} - \frac{N}{2}$$

$$\text{therefore } C = \frac{B}{a} - \frac{N}{2} \quad \ldots\ldots\ldots \quad (a)$$

Taking moments about the line of action of C, we find

$$T = \frac{B}{a} + \frac{N}{2} \quad \ldots\ldots\ldots \quad (b)$$

In order to demonstrate that the proposed method of measuring the eccentricity is a fair assumption, it is necessary to show that the general expressions for T and C satisfy the following fundamental principles:

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(I) That the sum of all the forces on the section shall equal zero. Algebraically adding C, T and N we have, from *Fig.* 1 :—

$$\left(\frac{B}{a}-\frac{N}{2}\right)-\left(\frac{B}{a}+\frac{N}{2}\right)+N=0$$

(II) That, when there is no direct force, T shall equal C.

When $N=0$; $C=\frac{B}{a}$ and $T=\frac{B}{a}$.

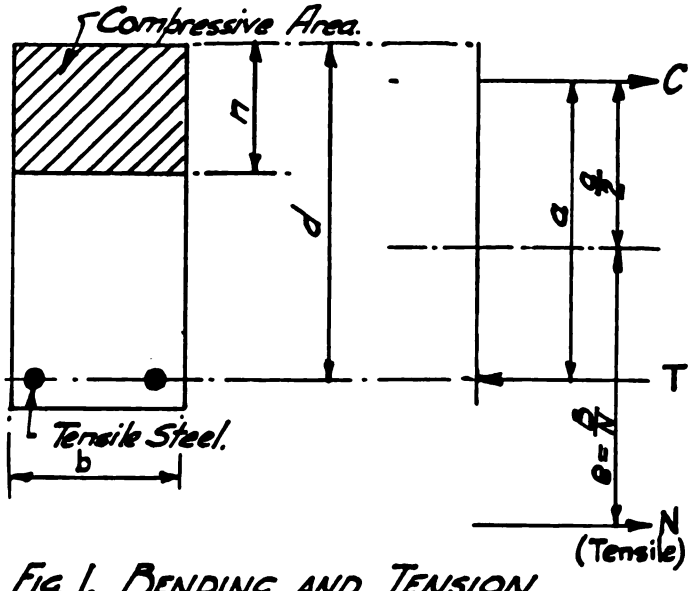

FIG. 1. BENDING AND TENSION.

These general formulæ (*a*) and (*b*) will now be applied to the design and analysis of rectangular sections, making those assumptions usual in reinforced concrete calculations and considering that N and B are known.

Case 1.—*To design a section with steel on one side only*, such that the stresses in concrete and steel shall not exceed given values c and t respectively.

It is first necessary to assume an effective depth ($=d$) for the section. Given the values of c and t, the depth of the neutral axis from the compression edge is given by

$$n=\frac{d}{1+\frac{t}{mc}} \quad \ldots\ldots\ldots \quad (1)$$

where m is the modular ratio, assumed equal to 15.
The lever arm is found from

$$a=d-\frac{n}{3} \quad \ldots\ldots\ldots \quad (2)$$

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The values of B, N, and a can now be substituted in formula (a) to give the total compression C, which also equals $\frac{bnc}{2}$

$$\text{whence } b = \frac{2C}{cn} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Evaluating (3) will determine the minimum breadth of section required to keep the concrete stress within the specified limit c. If this breadth is comparable with the assumed depth, then the section will hold good. If not, another value of d will have to be assumed and the procedure repeated. Having found a satisfactory depth, the amount of steel required can be determined from

$$A_T = \frac{T}{t}$$

$$\text{i.e. } A_T = \frac{C + N}{t} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

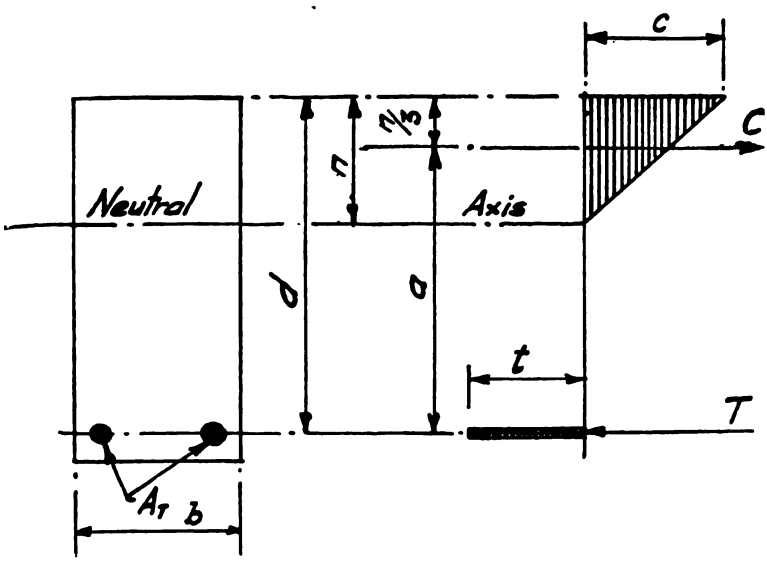

FIG. 2. SINGLE REINFORCED SECTION.

Example.—(a) Design a section for a rectangular beam where $B = 120{,}000$ in.-lbs. and $N = 5{,}000$ lbs. Stresses not to exceed 600 and 16,000 lbs. per sq. in.

Assume $d = 14{\cdot}5$ ins.

From (1) $n = 0{\cdot}36 \times 14{\cdot}5 = 5{\cdot}21$ ins.

,, (2) $a = 14{\cdot}5 - 1{\cdot}74 = 12{\cdot}76$ ins.

,, (a) $C = \frac{120{,}000}{12{\cdot}76} - \frac{5000}{2} = 6{,}900$ lbs.

,, (3) $b = \frac{2 \times 6{,}900}{600 \times 5{\cdot}21} = 4{\cdot}42$ ins.

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This value for the breadth is obviously too small. Hence re-work with $d = 12{\cdot}5$ and therefore $a = 11$ ins., $n = 4{\cdot}5$ ins.

From (*a*) $C = 8{,}400$ lbs.

,, (3) $b = 6{\cdot}22$ ins.

Make $b = 7$ ins., which is reasonable compared with a total depth of $12{\cdot}5 + 1{\cdot}5 = 14$ ins.

From (4) $A_T = (8{,}400 + 5{,}000)\frac{1}{16{,}000} = 0{\cdot}837$ ins. sq.

Example.—(*b*) A foot strip of slab is subjected to $B = 48{,}000$ in.-lbs. and $N = 2{,}000$ lbs. (tensile). Maximum stresses not to exceed 600 and 16,000 lbs. per sq. in. Design a suitable section.

From inspection it appears that a slab of effective depth 6·5 ins. would be satisfactory if the bending moment were acting alone. As N will relieve the compressive stress, assume a slab 7 ins. thick, i.e. $d = 6{\cdot}25$ ins.

Then $n = 2{\cdot}25$ ins. and $a = 5{\cdot}5$ ins. Hence $C = 7{,}720$ lbs.

In the case of a slab, the breadth as given by formula (3) should be approximately equal to, but not greater than, 12 ins. Substituting in (3)

$$b = \frac{2 \times 7{,}720}{600 \times 2{\cdot}25} = 11{\cdot}4 \text{ ins.}$$

which is close enough to 12 ins. to be economical.

Reinforcement required will be found from (4).

$A_T = 0{\cdot}606$ sq. ins. per foot width.

Case 2.—*To determine the stresses on a given section with steel on one face only.*

In this case A_T, b, and d are known. From inspection assume a value for n. Then, as in Case 1, a can be found from (2) and C from (*a*).

$$\text{Maximum stress in concrete is given by } c = \frac{2C}{bn} \quad . \quad . \quad . \quad (5)$$

$$\text{Maximum stress in steel is given by } t = \frac{C + N}{A_T} \quad . \quad . \quad . \quad (6)$$

With these values of c and t find what value of n is required by substitution in formula (1). If this value is approximately equal to the assumed value, the stresses obtained are satisfactory. If there is any discrepancy another value of n must be assumed and the problem re-worked. The second trial should give satisfactory results.

Example.—Give a section where $d = 18{\cdot}5$ ins., $b = 12$ ins., and $A_T = 1{\cdot}8$ sq. ins. Find maximum stresses when $B = 128{,}000$ in.-lbs. and $N = 4{,}000$ lbs. tension.

Assume $n = 6$ ins.; then $a = 16{\cdot}5$ ins. From (*a*) and (5): $c = 160$ lbs. per sq. in., and from (6): $t = 5{,}400$ lbs. per sq. in.

Substituting these values in (1) $n = 5{\cdot}7$ ins.

This does not show an appreciable difference from the assumed value of n. We will, however, assume a closer figure, say 5·8 ins.

Then $a = 16{\cdot}57$ ins., $c = 165$ lbs. per sq. in., and $t = 5{,}430$ lbs. per sq. in., while the value of n to satisfy these stresses is 5·8 ins. This shows that the first trial position of neutral axis was near enough, as the difference in the final stresses is very small.

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Case 3.—*To design a section with steel on both faces*; stresses not to exceed given values c and t.

From inspection of the problem, assume values for d and b. From formula (1) n can then be found.

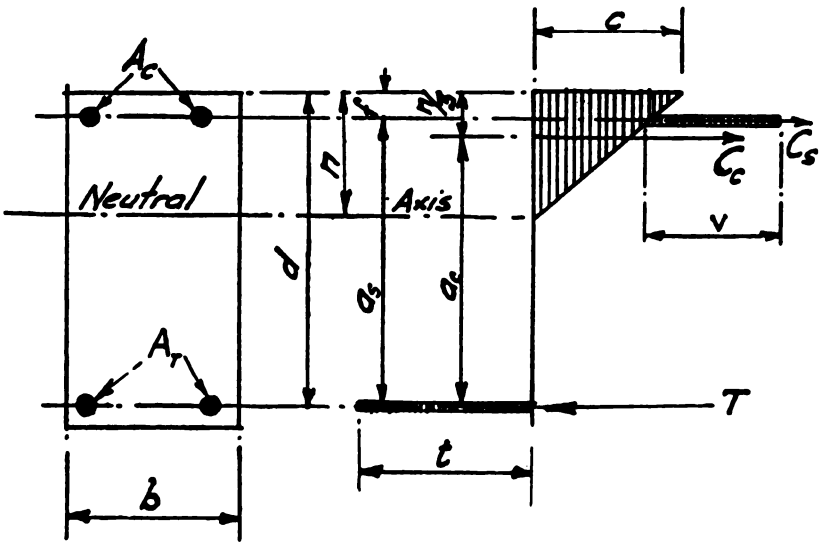

FIG. 3 DOUBLE REINFORCED SECTION.

With the conditions indicated in *Fig.* 3 the total compression is the sum of compressions due to steel and concrete, i.e. $C = C_s + C_c$.

If v = stress in compression steel * $v = \dfrac{c(m-1)(n-f)}{n}$ . . . (7)

and $C_s = A_c v$ which acts at a distance a_s from the tension steel where

$$a_s = d - f \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2a)$$

Due to concrete

$$C_c = \frac{bnc}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

which acts at a lever arm from the tension steel equal to

$$a_c = d - \frac{n}{3}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

The lever arm of the resultant compression will be

$$a = \frac{a_c C_c + a_s(C - C_c)}{C}$$

From formula (a) this lever arm also equals $a = \dfrac{B}{\dfrac{N}{2} + C}$.

* NOTE.—This is not the actual stress, but fourteen times the stress in concrete adjacent to the compression steel. The actual stress is $\frac{15v}{14}$ lbs. per sq. in., when $m = 15$.

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Equating these expressions and transposing we get

$$a_s C^2 - C_c(a_s - a_c) - \frac{N}{2}a_s + BC - \frac{NC_c}{2}(a_s - a_c) = 0$$

Therefore if we put

$$V = C_c(a_s - a_c) - \frac{N}{2}a_s + B \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

$$T = \frac{C_c N}{2}(a_s - a_c)$$

and $$U = 4a_s T$$

i.e. $$U = 2C_c N a_s(a_s - a_c) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

we have a quadratic of the form

$$a_s C^2 - VC - T = 0$$

therefore $$C = \frac{1}{2a_s}(V + \sqrt{V^2 + U}) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

Then $$C_s = C - C_c$$

And $$A_c = (C - C_c)\frac{1}{V} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

$$A_T = (C + N)\frac{1}{t} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Should the values of A_c and A_T be unreasonable, adjustment must be made to the assumed values of d and b. In the case where b and d are unalterable and A_c is greater than A_T, it is usual to put the same amount of steel in for compression as is required for tension. The limit of economical design is reached when the value of A_c given by (12) equals the value of A_T given by (4).

The working out of this case is greatly simplified if a_c is approximately equal to a_s, as then $a = a_c = a_s$, and C can be obtained directly from formula (a), after which proceed as above. An example should make the method clear.

Example.—With maximum stresses of 17,000 and 650 lbs. per sq. in., design a section to carry $B = 360{,}000$ in. lbs. and $N = 12{,}000$ lbs. Overall depth of section not to exceed 15 ins.

Assume section: 15 ins. × 7 ins., i.e. $d = 13{\cdot}5$ ins.

Then $n = 4{\cdot}93$ ins. and $a_c = 13{\cdot}5 - 1{\cdot}64 = 11{\cdot}86$ ins., and $a_s = 13{\cdot}5 - 1{\cdot}5 = 12$ ins.

It is seen that a_c and a_s are near enough equal for us to consider that $a = 12$ ins.

From (a) $C = \frac{360{,}000}{12} - \frac{12{,}000}{2} = 24{,}000$ lbs.

From (8) $C_c = \frac{7 \times 4{\cdot}93 \times 650}{2} = 11{,}200$ lbs.

From (7) $v = 650 \times 14 \times \frac{3{\cdot}43}{4{\cdot}93} = 6{,}340$ lbs. per sq. in.

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From (12) $A_c = \dfrac{24{,}000 - 11{,}200}{6{,}340} = 2{\cdot}02$ sq. ins.

From (4) $A_T = \dfrac{24{,}000 + 12{,}000}{17{,}000} = 2{\cdot}12$ sq. ins.

These values for the area of reinforcement are satisfactory.

This example will be worked again to show the applicability of the full method.

Find as above the values of a_c, a_s, C_c, and v

Substituting in (9): $V = 288{\cdot}5 \times 10^3$

and in (10): $U = 129 \times 10^6$

From (11): $C = \dfrac{[288{\cdot}5 + \sqrt{288{\cdot}5^2 + 129}]10^3}{24}$

$= 24{,}000$ lbs.

which is the result obtained by the approximate method. Beyond this stage the working will be the same.

Case 4.—*To determine the stresses on a given section with steel on both faces.*

This case is similar to Case 2 as regards assuming a reasonable value for the position of the neutral axis. Then with the same significance for the symbols as in Case 3:

$$a_c = d - \frac{n}{3} \quad \ldots \quad (2)$$

$$a_s = d - f \quad \ldots \quad (2a)$$

If

$$k_c = \frac{bn}{2} \quad \ldots \quad (13)$$

$$C_c = k_c c$$

and if

$$k_s = \frac{A_c(m-1)(n-f)}{n} \quad \ldots \quad (14)$$

$$C_s = k_s c$$

$$C = k_c c + k_s c = c(k_c + k_s)$$

Lever arm of resultant compression $= \dfrac{ca_c k_c + a_s k_s c}{c(k_c + k_s)}$

i.e.

$$a = \frac{a_c k_c + a_s k_s}{k_c + k_s} \quad \ldots \quad (15)$$

To determine the resultant compression, evaluate expression (15) and substitute in

$$C = \frac{B}{a} - \frac{N}{2} \quad \ldots \quad (a)$$

Then

$$c = \frac{C}{k_c + k_s} \quad \ldots \quad (16)$$

and

$$t = \frac{C + N}{A_T} \quad \ldots \quad (4)$$

With these stresses check the assumed value of n.

Example.—Given $N = 6{,}000$ lbs. (tension); $B = 120{,}000$ in. lbs. Section

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12 ins. by 6 ins., reinforced with two $\frac{7}{8}$-in. bars top and bottom. To determine maximum stresses: $f = 1\frac{1}{4}$ ins.

Assume $n = 3{\cdot}75$ ins.

Then $a_c = 10{\cdot}75$ ins., $a_s = 10{\cdot}75$ ins.

and $k_c = 11{\cdot}25$; $k_s = 11{\cdot}2$.

Since $a_c = a_s$; $a = 10{\cdot}75$ ins.

From (a) $C = \dfrac{120{,}000}{10{\cdot}75} - \dfrac{6000}{2} = 8{,}150$ lbs.

From (16) $c = \dfrac{8{,}150}{11{\cdot}25 + 11{\cdot}2} = 363$ lbs. per sq. in.

From (4) $t = \dfrac{8{,}150 + 6{,}000}{1{\cdot}2} = 11{,}800$ lbs. per sq. in.

Value of n to give these stresses:

$$= \frac{12}{1 + \dfrac{11{,}800}{15 \times 363}} = 3{\cdot}8 \text{ ins.,}$$

which is near enough assumed value.

BENDING AND COMPRESSION.

When the direct force N is a thrust, the forces acting on the section are as shown in *Fig.* 4.

The same symbols will be used throughout as for the cases involving direct tension and bending.

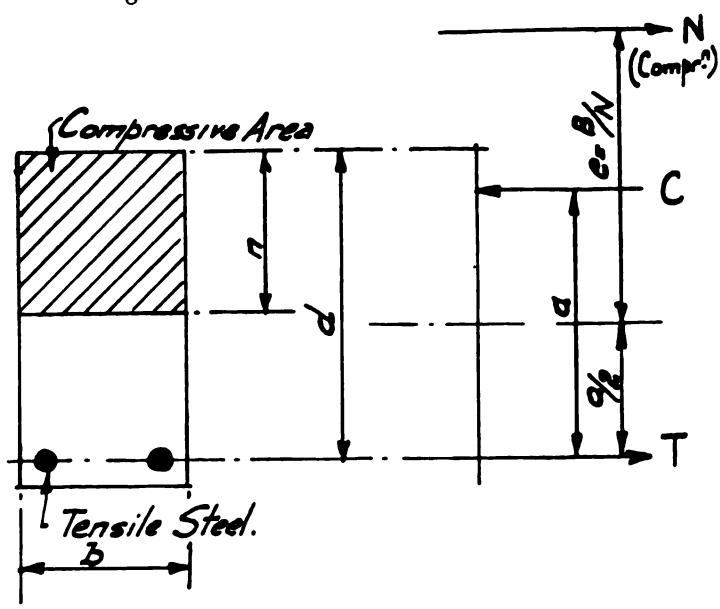

FIG. 4. BENDING & COMPRESSION.

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Taking moments about the line of action of T:—

$$Ca = N\left(e + \frac{a}{2}\right)$$

$$C = \frac{Ne}{a} + \frac{N}{2}$$

$$C = \frac{B}{a} + \frac{N}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (c)$$

Taking moments about the line of action of C we similarly find that

$$T = \frac{B}{a} - \frac{N}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (d)$$

The analysis and design of sections subjected to combined bending and direct compression will be very similar to that for combined bending and direct tension. The expressions for the various stresses, etc., will, however, be based on the two general formulæ (c) and (d).

No examples will be worked out in this part, the procedure being practically the same as for the corresponding cases for direct tension.

Case 1.—*To design a section with steel on one face only,* given the maximum permissible stresses.

Assume a trial value for d.

Then

$$n = \frac{d}{1 + \frac{t}{cm}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

and

$$a = d - \frac{n}{3} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$C = \frac{B}{a} + \frac{N}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (c)$$

Breadth required:

$$b = \frac{2C}{cn} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

If b is satisfactory compared with d, the area of steel required is given by

$$A_T = \frac{C - N}{t} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Case 2.—*To determine stresses on a given section with steel on one face only.*

Assume a trial value for n from inspection.

Then

$$a = d - \frac{n}{3} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$C = \frac{B}{a} + \frac{N}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (c)$$

$$c = \frac{2C}{bn} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

$$t = \frac{C - N}{A_T} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6a)$$

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Find from $n = \dfrac{d}{1 + \dfrac{t}{cm}}$ the value of n required to satisfy the stresses given by (5) and (6) and compare with the trial value.

Case 3.—*To design a section with steel on both faces*; stresses not to exceed given stresses c and t.

Assume a section; that is, b and d. Find position of neutral axis from

$$n = \frac{d}{1 + \dfrac{t}{cm}} \quad \ldots \quad (1)$$

Determine the following values:—

$$v = \frac{c(m-1)(n-f)}{n} \quad \ldots \quad (7)$$

$$a_c = d - \frac{n}{3} \quad \ldots \quad (2)$$

$$a_s = d - f \quad \ldots \quad (2a)$$

$$C_c = \frac{bnc}{2} \quad \ldots \quad (8)$$

$$V = C_c(a_s - a_c) + B + \frac{a_s N}{2} \quad \ldots \quad (9a)$$

$$U = 2C_c a_s N(a_s - a_c) \quad \ldots \quad (10)$$

To find total compression, substitute in

$$C = \frac{V + \sqrt{V^2 - U}}{2a_s} \quad \ldots \quad (11a)$$

Then amount of reinforcement required will be found from

$$A_c = \frac{C - C_c}{v} \quad \ldots \quad (12)$$

$$A_T = \frac{C - N}{t} \quad \ldots \quad (4a)$$

The observations following the case corresponding to this for direct tension apply equally well to this case.

Case 4.—*To determine the stresses on a given section with steel on both faces.*

Assume a trial value for n.

Then

$$a_c = d - \frac{n}{3} \quad \ldots \quad (2)$$

$$a_s = d - f \quad \ldots \quad (2a)$$

$$k_c = \frac{bn}{2} \quad \ldots \quad (13)$$

$$k_s = \frac{A_c(m-1)(n-f)}{n} \quad \ldots \quad (14)$$

$$a = \frac{a_c k_c + a_s k_s}{k_c + k_s} \quad \ldots \quad (15)$$

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Substitute these values in

$$C = \frac{B}{a} + \frac{N}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (a)$$

$$c = \frac{C}{k_c + k_s} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

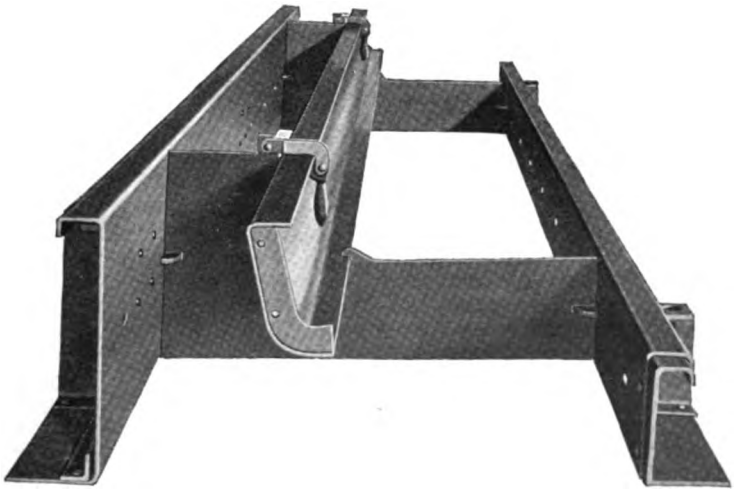
$$t = \frac{C - N}{A_T} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4a)$$

With these stresses check the trial position of the neutral axis.

In the above cases it has been assumed that the values of the bending moment and direct force are both given. If, however, the value of the direct force and its line of action are given, it will be necessary to find the distance (say $= e$) from this line of action to the axis midway between the lines of action of T and C. Then the bending moment B will be the product of the direct force N and 'e', and problems can be worked out as detailed above.

A New Kerb and Gutter Form.

THE simplicity of the well-known "Metaform" steel shuttering has been incorporated in a new steel kerb form just put on the market by Messrs. A. A. Byrd & Co., of 11, Queen Victoria Street, E.C.4. The saving in labour and materials which is effected by steel forms compared with wood when they can be used many times over is so



great that this device warrants the attention of road engineers laying concrete kerbing. The method of forming the kerb and gutter is clearly shown in the illustration. The forms are supplied in 10-ft. lengths. The side rails are made of 10-gauge blue annealed steel, and are readily connected and securely locked to the division plates. The face-rail is placed in position by engaging its lower edge under the hook projection in the division plate, and locked by the clamp. The form consists of two side-rails, a division plate, and a face-rail, which are securely locked together to form a rigid shuttering. Abutting forms are locked together by heavy end boxes. Division plates are secured to the side-rails by hook clamps inserted through slots in the side-rails at 12-in. centres, and are fastened and unfastened by a blow with a hammer on an eccentric wedge. This permits short lengths of kerb to be constructed in any multiple of 12 ins., so that kerbs which are worn or graduated may be easily replaced. Steel stakes are used to hold the forms in position, and they are held rigid in stake boxes by steel taper wedges.

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Questions and Answers relating to Reinforced Concrete.

Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.—ED.

Effect of Temperature on Strength of Concrete.

QUESTION.—I have read in your columns a number of articles on the effect the proportion of water, sodium silicate, oils, etc., have on the ultimate strength of concrete, but am unaware of one dealing with the effect of humidity and temperature during the maturing time. That "drying out" weakens concrete is axiomatic, but has the balance of cause and effect been scientifically studied? Can better results be obtained from steam curing than suitably safeguarded air curing?—H. B.

ANSWER.—The effect of humidity and temperature during the maturing period of concrete is as follows: During this period the setting of the concrete requires water to be present for the chemical reactions to take place. It therefore follows that the concrete should be kept moist for the whole of its maturing time. In fact, concrete which is thoroughly dried ceases to gain in strength after the time of its drying. As regards the temperature, the higher the temperature at which the maturing takes place the more rapidly is the curing effected; and not only this, but stronger concrete is produced than is produced when maturing at a low temperature. In other words, it is not merely that the setting is speeded up, because the increased strength of the concrete produced when setting at a high temperature over and above the strength when setting at a low temperature will subsequently be maintained when both concretes are allowed to continue their maturing under normal conditions. Roughly speaking, a concrete which gives 300 lbs. per sq. in. tension strength in 7 days after maturing at 60 deg. will give about 400 lbs. per sq. in. at the same period if it matures at 174 deg., and about 500 lbs. if it matures at 212 deg. Care must be taken that the high-temperature heat is a wet and not a dry one, and this is best ensured by blowing steam into the maturing chamber. It is also important that the changes of temperature are not made too rapidly, as otherwise the outer surface of the concrete article gets warm and sets before the inside. This is particularly important in the case of members having a considerable thickness.—O. F.

Concrete Blocks.

QUESTION.—I would like to put a question with reference to the making of concrete blocks, lintels, steps, etc. The aggregate used is whin chips (2 parts ½-in. and 3 parts ¼-in. to dust) to 1 part Portland cement. The approximate mechanical analysis of the ¼-in. to dust whin is 50 per cent. through $\frac{1}{16}$-in. mesh sieve and 25 per cent. of the remainder through a 30-mesh sieve. From actual experience I find a good percentage of blocks made are very slow in setting, while others set all right. The blocks that do not set are so soft that a knife can be pushed in easily 4 or 5 days after making, and at the end of three weeks or so they fall to pieces when handling. There are many makers of blocks using steel furnace slag as an aggregate, using the same brand of cement, and turning out splendid blocks. —A. D.

ANSWER.—We would suggest that what is wrong with the concrete is partly that there is too little ½-in. material and too much small material, and, furthermore, that the dust is being used, which is always dangerous. We would recommend a mixture somewhat as follows: 3 parts ½-in. to 2 parts of ¼-in. down and 1 part of cement. All material passing through a $\frac{1}{16}$-in. mesh should be rejected, so that the ¼-in. material will really be between ¼-in. and $\frac{1}{16}$-in. The reason that the steel furnace slag is turning out good blocks is probably because the crushed slag contains practically no dust. If our correspondent will make some tests on the lines indicated and advise us of the results we should be glad to give him any further assistance possible.—O. F.

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Relative Economy of Different Classes of Concrete.

By Sir Owen Williams, K.B.E., M.Inst.C.E.

In the course of a lecture delivered before the Institution of Civil Engineers last month, entitled "The Relative Economy of Different Classes of Concrete for Reinforced Concrete Work," Sir E. Owen Williams, K.B.E., B.Sc., M.Inst.C.E., said the first use of concrete was in mass, and, as that word signified, weight was intrinsically valuable. The thickness of a retaining wall could be no less, whatever the quality of the concrete; hence engineers came to understand that the necessary amount of cement in mass concrete was only such as would prevent the concrete returning to original ballast. In that connection experimental work was devoted to determining mixes of ballast and sand demanding the least amount of cement. But reinforced concrete arrived, and no longer was the weight of concrete of any value; indeed it became a burden to be reduced. New values came into prominence. Compressive strength, adhesion to steel, watertightness, became of primary importance. Increased values in any of these enabled a reduction of sizes and were, therefore, much to be desired. That was a different view-point of concrete. In one case it was a question of determining the least amount of cement for a given and fixed mass of concrete; the other was a case of getting the maximum compressive strength, adhesion to steel, and watertightness with the least weight of concrete.

Some years ago he made a series of experiments to determine the influence of the percentage of cement on these qualities of concrete, assuming equal care in grading of aggregates for all mixtures. From the results of his experiments he came to the conclusion that all properties of concrete varied directly with the percentage of cement in the finished concrete. For example, a 4 : 2 : 1 concrete had approximately 510 lbs. of cement per yard cube of concrete, whereas in a 2 : 1 : 1 concrete the cement weight was 860 lbs. per yard cube, and if it were reasonable to take 510 lbs. per sq. in. compression on a 4 : 2 : 1 concrete experiment showed that it would be equally reasonable to take 860 lbs. compression on a 2 : 1 : 1 concrete.

The significance of that was that in a column of any size within limits a ton of cement would give the same bearing capacity whatever the size of column. Thus, there was no ineconomy in rich mixtures—in fact, there was the reverse. Using the higher percentage of cement paradoxically resulted in no increase in the quantity of cement, but it reduced the quantities of aggregates, moulds, and weight. Adhesion to the steel was increased—a most desirable result, as it was that adhesion which ultimately determined the strength of the structure. Also, whereas a 4 : 2 : 1 mixture was seldom watertight, a 2 : 1 : 1 concrete was seldom not watertight, resulting in increased permanence of construction because of less risk of steel corrosion.

Indeed, the cement in reinforced concrete might be likened to the reinforcement. The reinforcement was calculated by itself, and resulted in so many tons. The cement could also be calculated in so many tons, and then the designer would only have to make up his mind what bulkage he had to add in the way of ballast and sand to make the size of members practicable.

In brief, mass concrete must contain a predetermined and fixed weight of aggregates with as little as possible of cement, whereas reinforced concrete must contain a predetermined and fixed weight of cement with as little as possible of aggregates.

In the majority of structures built in reinforced concrete the concrete had very hesitatingly and timidly been made a little better than mass concrete, being fallaciously regarded from the same standpoint, i.e. from the standpoint of saving cement, whereas in reinforced concrete it was a case of saving ballast and sand, the cement being a fixed tonnage.

It would be asked, Where should this increased percentage of cement end? Why not use cement only? The answer was that the practicable sizes of members demanded a greater volume than would

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be economically supplied by the calculated weight of cement, and to produce those sizes ballast and sand were necessary, although it was quite possible that in certain structures where loads were considerable the amount of ballast and sand would be very small.

The following tables showed relative costs of slabs to carry the same load on the same span and columns carrying the same load :—

Slabs.		Relative Cost Per cent.
6 in. Slab	in 4 : 2 : 1	100
5¼ in. ,,	,, 3 : 1½ : 1	97
4⅝ in. ,,	,, 2 : 1 : 1	97

(All carrying same super load on same span.)

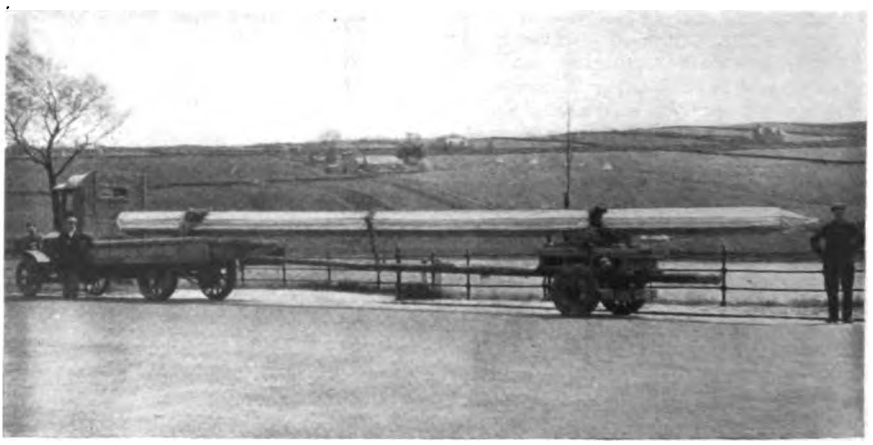
Columns.		Relative Cost. Per cent.
12 in. Columns	4 : 2 : 1	100
10·7 in. ,,	3 : 1½ : 1	88
9·5 in. ,,	2 : 1 : 1	76

In the second table each of the columns carried the same load, neglecting reduction of weight by use of less concrete in floors and slabs, and the table was based upon reasonable allowable stresses. Both tables showed the distinct advantage of the better concrete.

He had found that in actual work the practicable sizes of beams and slabs demanded a concrete somewhere between 3 : 1½ : 1 and 2 : 1 : 1 mixtures. In practical manipulation there was no comparison between the richer mixes and the poorer. For example, in the 4 : 2 : 1 mixture 10 per cent. too much water removed 20 per cent. of its strength, whereas in the richer mixture of 2 : 1 : 1 10 per cent. too much water only removed 10 per cent. of strength. Further, in the richer mixture, because of the increased percentage of cement acting as a lubricant, there was not the same temptation to use an excess of water. The result of that, as practical experience had shown, was far greater uniformity of concrete in the case of the richer mixtures.

He found a 4 : 2 : 1 mixture almost universally used, and could only think that was due to an inappreciation of the changed conditions from mass concrete to reinforced concrete, i.e. a fallacy had been born and was being perpetuated.

Transporting a 48-ft. Reinforced Concrete Pile.



[The above illustration shows a reinforced concrete pile, 48 ft. long by 16 in. section, ready for transport. This is one of a batch of thirty which were transported in this manner a distance of sixteen miles by Mr. W. J. Anderson, of Aberdeen.]

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Sawdust as an Aggregate.

In the course of a paper read before the Society of Engineers last month, Mr. Gerald O. Case dealt at length with the subject of the use of sawdust and small wood chips as a concrete aggregate. He pointed out that the difficulty to be overcome was the change in volume of the wood after it was embedded in cement, with a consequent loss of adhesion; if such aggregate were thoroughly saturated before use it shrunk as the concrete dried, while if it were mixed dry it absorbed moisture from the mix and injured the set of the cement. After a considerable amount of research a process of mineralisation was discovered which prevented the wood from expanding or contracting, and at the same time rendered it fireproof.

The following tests were made in England in 1925. In all these tests mineralised sawdust and cement were measured by volume. With 2 : 1 mixes, 23·4 per cent. of water was used, with 3 : 1 mixes 25·5 per cent., and with 6 : 1 mixes, 29·3 per cent. of water. The water content was calculated by weight of the dry materials. The briquettes were cured in air, ordinary Portland cement being used.

Parts by volume min. sawdust and cement.	Age of briquettes.	No. of briquettes tested.	Aver. tensile strength, lbs. per sq. in.	Grade of min. sawdust used.
2 : 1	7 days	3	387	Coarse common deal sawdust.
2 : 1	14 ,,	3	443	,,
2 : 1	28 ,,	3	513	,,
2 : 1	3 months	3	520	,,
3 : 1	7 days	3	367	,,
3 : 1	14 ,,	3	410	,,
3 : 1	28 ,,	3	488	,,
3 : 1	3 months	3	500	,,
2 : 1	7 days	3	427	Rather fine common deal sawdust.
2 : 1	14 ,,	3	467	,,
2 : 1	28 ,,	3	537	,,
2 : 1	3 months	3	548	,,
3 : 1	7 days	3	313	,,
3 : 1	14 ,,	3	380	,,
3 : 1	28 ,,	3	443	,,
3 : 1	3 months	3	450	,,
6 : 1	7 days	3	257	,,
6 : 1	14 ,,	3	263	,,
6 : 1	28 ,,	3	310	,,
6 : 1	3 months	3	340	,,

In making these tests it was noticed that the briquettes, especially those made of lean mixes, became considerably elongated before fracturing. Since the date of these tests, a large number of 3 : 1 briquettes had been made, and in a number of cases it was found that, when they were tested in a standard Avery machine, it was impossible to break them as they elongated so much that the arm of the machine came to rest before the briquette was broken. In all the tests the mineralised sawdust was measured by volume, and the results obtained were therefore not strictly comparable with the 3 : 1 sand cement mixture where the quantities were measured by weight. Taking the weight of standard sand at 100 lbs. per cu. ft. and cement measured loose at 90 lbs. per cu. ft., then 3 : 1 by weight mixture would be equal to 2·7 : 1 mixture by volume. Therefore a 3 : 1 mixture of mineralised sawdust and cement measured by volume contained less cement than a 3 : 1 by weight sand and cement mixture.

The following table showed the results of tests made in 1925 to ascertain the compressive strength of cubes made with mineralised sawdust and ordinary Port-

Proportion of min. sawdust to Portland cement by volume.	No. of cubes tested.	Age of cubes.	Aver. compressure strength in lbs. per sq. in.	Grade of min. sawdust used
2 : 1	3	7 days	1,967	Coarse common deal sawdust.
2 : 1	3	14 ,,	2,167	,,
2 : 1	3	28 ,,	2,583	,,
2 : 1	3	3 months	2,967	,,
3 : 1	3	7 days	1,333	,,
3 : 1	3	14 ,,	1,500	,,
3 : 1	3	28 ,,	1,733	,,
3 : 1	3	3 months	2,167	,,

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land cement. In making the cubes, 23·4 per cent. of water was used, the water content being calculated by the weight of the dry materials. All the cubes were cured in air and broken at 7, 14 and 28 days, and 3 months.

Subsequently further cubes (3 : 1 by volume) were made with a better grade of mineralised sawdust, and these showed an average tensile strength of 2,483 lbs. at 28 days. In making the above tests it was noticed that the specimens in compression behaved differently from ballast and sand concrete, inasmuch as they compressed without disintegrating after the maximum compression stress had been reached, the gauge needle often remaining stationary at the maximum reading when the specimens were being still further compressed.

Abrasion tests showed that concrete made with mineralised sawdust as an aggregate was more durable when subjected to severe wear than concrete made with a sand aggregate. That was probably due to the fact that as soon as sand cement concrete commenced to wear the particles came out whole, and not only added to the abrasive action but the little holes in the concrete made a rough surface and allowed the abrasive action to be carried on very rapidly; while mineralised sawdust did not come out as a whole, but slowly wore away. The abrasive action also resulted in the formation of very fine particles of both the cement and mineralised sawdust, which latter in themselves did not assist attrition.

A slab made of two parts of mineralised sawdust and one part of cement was subjected to the fire test as required for building materials by the Bureau of Buildings of the City of New York. One side of the slab was consecutively subjected to an average temperature of 1,083° F. for the first half hour, and of 1,738° F. for the second half hour. Water was then applied to the fire-side of the slab immediately after it had been taken from the furnace, and when it had been cooled in air it was found that the portion made with mineralised sawdust and Portland cement (6 to 1 by volume) and exposed to the fire was discoloured and rendered friable to the maximum depth of ¾ of an inch, the balance of the slab being sound and unaffected by the test. Absorption tests showed that mixes of mineralised sawdust and cement in proportions varying from 2 : 1 to 4 : 1 absorbed about 9·5 lbs. of water per cubic foot after immersion in water to a depth of 1½ in. after 48 hours.

Tests for contraction showed a percentage of from 0·025 to 0·98 per cent. on a rod 2 ft. long by 2 in. square made of a 3 : 1 mixture six days after it had been made.

Transverse bending tests made on a slab measuring 12 in. by 24 in. by 2 in., made of a 2 : 1 mix and supported for the full width across a span of 22 in., loaded at midspan, showed deflections ranging from nil with a load of 23 lbs. to 0·0436 lb. with a load of 789 lbs., the modulus of rupture being 537 lbs. per sq. in.

Tests for the adhesion of steel rods embedded in sawdust concrete were made by embedding ½-in. square bars in the centre of blocks 3 in. square by 6 in. long made of a 2 : 1 mixture. The actual load at which the bars pulled averaged 2,700 lbs., or 225 lbs. per sq. in. of adhesive area. The specimens were 28 days old when tested.

A freezing test was made with blocks composed of a 4 : 1 mix of mineralised sawdust and Portland cement immersed in water for six hours and frozen through for 24 hours; then dried on a hot stove; again submerged in water for three hours and placed in a temperature of 7 deg. below zero; then placed on the white hot coal inside a stove and kept there for 15 minutes; and then plunged into ice-cold water. On examination there were no signs of disintegration, but charring to a depth of $\frac{1}{8}$th of an inch on the side which rested against the fire.

Sawdust-concrete had only half the weight of ordinary concrete; it could be nailed or screwed; it could be sawn; and it could be polished.

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Standard Specification for Reinforced Concrete.

AFFILIATED committees of the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering Association, the American Concrete Institute, and the American Portland Cement Association have for some years been engaged on the preparation of a Standard Specification for Concrete and Reinforced Concrete. This joint committee has had the assistance and advice of a large number of leading authorities on the subject, and has now published the results of its labours in the Proceedings of the Twenty-Seventh Annual Meeting of the Society for Testing Materials.

In the following abstract of the recommended Specification we have omitted many paragraphs and condensed others.

MATERIALS.

Portland Cement.

5. Portland cement shall conform to the Standard Specifications for Portland Cement.

Fine Aggregate.

6. Fine aggregate shall consist of sand or other approved inert materials with similar characteristics, or a combination thereof, having clean, hard, strong, durable, uncoated grains and free from injurious amounts of dust, lumps, soft or flaky particles, shale, alkali, organic matter, loam or other deleterious substances.

7. Fine aggregate shall range in size from fine to coarse within the limits indicated in Table I.

TABLE I.—GRADING OF FINE AGGREGATE.

	Per cent. by weight.
Passing through No. 4 sieve	not less than (*a*)
Passing through No. 50 sieve	not more than (*b*)
	not less than (*c*)
Weight removed by decantation . . .	not more than (*d*)

The grading requirements will vary with the type of work and characteristics of materials available in any given locality. Wherever practicable the following values should be inserted: (*a*) = 85 per cent.; (*b*) = 30 per cent.; (*c*) = 10 per cent.; (*d*) = 3 per cent.

10. Fine aggregate shall be of such quality that mortar briquettes, cylinders or prisms, consisting of one part by weight of Portland cement and three parts by weight of fine aggregate will show a tensile or compressive strength at ages of 7 and 28 days not less than . . . . . per cent. of that of 1 : 3 standard sand mortar of the same plasticity made with the same cement.

Coarse Aggregate.

13. Coarse aggregate shall consist of crushed stone, gravel, or other approved inert materials with similar characteristics, or combinations thereof, having clean, hard, strong, durable, uncoated particles free from injurious amounts of soft, friable, thin, elongated or laminated pieces, alkali, organic or other deleterious matter.

14. Coarse aggregate shall range in size from fine to coarse within the limits given in Table II.

TABLE II.—SIZE AND GRADING OF COARSE AGGREGATE.

Passing		Per cent. by weight.
——* in. sieve (maximum size) . .	not less than	95
——* in. sieve (intermediate size)	not less than	——*
	not more than	——*
No. 4 sieve	not more than	10
No. 8 sieve	not more than	5

* Insert sizes and percentages required with regard to materials available. The following table indicates desirable gradings for coarse aggregates of certain nominal maximum sizes.

Nominal Maximum Size of Aggregate in.	Percentage by Weight passing through Standard Sieves with Square Openings.						Percentage Passing not more than	
	3 in.	2 in.	1½ in.	1 in.	¾ in.	½ in.	No. 4 Sieve	No. 8 Sieve
3 . .	95	—	40–75	—	—	—	10	5
2 . .	—	95	—	40–75	—	—	10	5
1½ . .	—	—	95	—	40–75	—	10	5
1 . .	—	—	—	95	—	—	10	5
¾ . .	—	—	—	—	95	—	10	5
½ . .	—	—	—	—	—	95	10	5

REINFORCEMENT.

21. Metal reinforcement shall conform to the requirements of the Standard Specifications.

22. Reinforcement bars shall conform to the areas and equivalent sizes shown in Table III.

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TABLE III.—SIZES AND AREAS OF REINFORCEMENT BARS.

Size of Bar, in.	Area, sq. in.	
	Round Bar.	Square Bar.
3/8	0·110	—
1/2	0·196	0·250
5/8	0·306	—
3/4	0·441	—
7/8	0·601	—
1	0·785	1·000
1 1/8	—	1·265
1 1/4	—	1·562

23. An approved deformed bar shall be one that will develop a bond at least 25 per cent. greater than that of a plain round bar of equivalent cross-sectional area. The areas of deformed bars shall be determined by the minimum cross-section thereof.

PROPORTIONING.

27. The unit of measure shall be the cubic foot. Ninety-four pounds of cement shall be considered as one cubic foot.

28. The method of measurement shall be such as to secure the specified proportions in each batch. The aggregates shall be measured separately by volume weight. In volume measurement the fine aggregate and the coarse aggregate shall be measured loose, as thrown into the measuring device, and struck off. The water shall be so measured as to ensure the desired quantity in successive batches.

29. Concrete shall be mixed in the proportions indicated in Table IV.

TABLE IV.—PROPORTIONS FOR CONCRETE.

Part of Structure.	Proportions by Volume.			Concrete Strength Assumed as Basis for Design. Lbs. per sq. in. at 28 Days.
	Cement.	Fine Aggregate.	Coarse Aggregate.	
Footings	—	—	—	—
Beams	—	—	—	—
Girders	—	—	—	—
Slabs	—	—	—	—
Columns	—	—	—	—
. . .	—	—	—	—
. . .	—	—	—	—
. . .	—	—	—	—

Frequent tests shall be made throughout the work to determine whether the concrete produced by the proportions given in Table IV is of the quality specified. These tests shall be made at the expense of the owner. The contractor shall provide such facilities as may be practicable for carrying out the tests, and shall co-operate in every way to the end that concrete of the desired quality shall be obtained.

CONSISTENCY OF CONCRETE.

30. The quantity of water used shall be the minimum necessary to produce concrete of a workability required by the Engineer. Attention is called to the fact that increased workability may be obtained by decreasing the quantity of coarse aggregate in the batch, without increasing the quantity of mixing water. The consistency of the concrete shall be measured by the slump test. The slump for the different types of concrete shall not be greater than that specified by the Engineer. The consistency shall be checked from time to time during the progress of the work.

MIXING.

31. The mixing shall be done in a batch mixer of approved type which will ensure a uniform distribution of the materials throughout the mass, so that the mixture is uniform in colour and homogeneous. The mixer shall be equipped with suitable charging hopper, water storage, and a water-measuring device controlled from a case which can be kept locked and so constructed that the water can be discharged only while the mixer is being charged. It shall also be equipped with an attachment for automatically locking the discharge lever until the batch has been mixed the required time after all materials are in the mixer. The entire contents of the drum shall be discharged before recharging. The mixer shall be cleaned at frequent intervals while in use. The volume of the mixed material per batch shall not exceed the rated capacity of the mixer.

32. The mixing of each batch shall continue not less than one minute after all the materials are in the mixer, during which time the mixer shall rotate at a peripheral speed of about 200 ft. per minute.

33. When hand mixing is authorised it shall be done on a water-tight platform.

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The cement and fine aggregate shall first be mixed dry until the whole is of a uniform colour. The water and coarse aggregate shall then be added and the entire mass turned at least three times, or until a homogeneous mixture of the required consistency is obtained.

34. The retempering of concrete or mortar which has partially hardened, that is, remixing with or without additional cement, aggregate, or water, will not be permitted.

DEPOSITING CONCRETE.

Depositing in Air.

35. Before beginning a run of concrete, hardened concrete and foreign materials shall be removed from the inner surfaces of the mixing and conveying equipment.

36. Before depositing concrete, débris shall be removed from the space to be occupied by the concrete ; forms shall be thoroughly wetted (except in freezing weather) or oiled. Reinforcement shall be thoroughly secured in position and approved by the engineer.

37. Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable by methods which shall prevent the separation or loss of the ingredients. It shall be deposited in the forms as nearly as practicable in its final position to avoid rehandling. It shall be so deposited as to maintain, until the completion of the unit, a plastic surface approximately horizontal. Forms for walls or other thin sections of considerable height shall be provided with openings or other devices that will permit the concrete to be placed in a manner that will avoid accumulations of hardened concrete on the forms or metal reinforcement. Under no circumstances shall concrete that has partially hardened be deposited in the work.

38. When concrete is conveyed by chuting, the plant shall be of such size and design as to ensure a practically continuous flow in the chute. The angle of the chute with the horizontal shall be such as to allow the concrete to flow without separation of the ingredients. An angle of 27 degrees, or one vertical to two horizontal, is the minimum slope which is considered permissible. Chuting through a vertical pipe is satisfactory when the lower end of the pipe is maintained as nearly as practicable to the surface of deposit, and the pipe full. The delivery end of the chute shall be as close as possible to the point of deposit. When the operation is intermittent the spout shall discharge into a hopper. The chute shall be thoroughly flushed with water before and after each run ; the water used for this purpose shall be discharged outside the forms.

39. Concrete, during and immediately after depositing, shall be thoroughly compacted by means of suitable tools. For thin walls or inaccessible portions of the forms, where rodding or forking is impracticable, the concrete shall be assisted into place by tapping or hammering the forms opposite the freshly-deposited concrete. The concrete shall be thoroughly worked around the reinforcement and embedded fixtures and into the corners of the forms.

40. Water shall be removed from excavations before concrete is deposited, unless otherwise directed by the engineer. Any flow of water into the excavation shall be diverted through proper side drains to a sump, or be removed by other approved methods which will avoid washing the freshly-deposited concrete. Water vent pipes and drains shall be filled by grouting or otherwise, after the concrete has thoroughly hardened.

41. Exposed surfaces of concrete shall be protected from premature drying for a period of at least seven days after being deposited.

42. Concrete when deposited shall have a temperature of not less than 40 deg. F. nor more than 120 deg. F. In freezing weather suitable means shall be provided for maintaining the concrete at a temperature of at least 50 deg. F. for not less than 72 hours after placing, or until the concrete has thoroughly hardened. Salt, chemicals, or other foreign materials shall not be mixed with the concrete for the purpose of preventing freezing, unless approved by the engineer.

43. Concrete shall be deposited continuously and as rapidly as practicable until the unit of operation is completed. Construction joints at points not provided for in the plans shall be made in accordance with the provisions in Section 69.

44. Before depositing new concrete on or against concrete which has set, the forms shall be re-tightened, the surface of the set concrete shall be roughened, thoroughly cleaned of foreign matter and laitance, and saturated with water. The new concrete placed in contact with hard-

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ened or partially hardened concrete shall contain an excess of mortar to ensure bond. To ensure this excess mortar at the juncture of the hardened and the newly-deposited concrete the cleaned and saturated surfaces of the hardened concrete, including vertical and inclined surfaces, shall first be slushed with a coating of neat cement grout against which the new concrete shall be placed before the grout has attained its initial set.

Rubble.

45. Rubble aggregate shall be thoroughly embedded in the concrete. The individual stones shall not be closer to any surface or adjacent stone than the maximum size of the coarse aggregate plus 1 in. Each successive layer of concrete shall be keyed in accordance with the provision in Section 69.

46. Cyclopean aggregate shall be thoroughly embedded in the concrete; no stone shall be closer to a finished surface than 1 ft., nor closer than 6 in. to any adjacent stone. Stratified stone shall be laid on its natural bed.

Depositing under Water.

Concrete shall be deposited by a method that will prevent the washing of the cement from the mixture, minimise the formation of laitance, and avoid flow of water until the concrete has fully hardened. Concrete shall be placed so as to minimise segregation of materials. Concrete shall not be placed in water having a temperature below 35 deg. F.

48. Concrete to be deposited under water shall contain 7 cu. ft. or more of Portland cement per cubic yard of concrete in place.

49. Cofferdams shall be sufficiently tight to prevent flow of water through the space in which the concrete is to be deposited. Pumping will not be permitted while concrete is being deposited, nor until it has fully hardened.

50. Concrete shall be deposited continuously, keeping the top surface as nearly level as possible until it is brought above the water or to the required height. The work shall be carried on with sufficient rapidity to prevent the formation of layers.

51. One of the following methods shall be used for depositing concrete under water:

(*a*) *Tremie.*—The tremie shall be water-tight and sufficiently large to permit a free flow of concrete. It shall be kept filled at all times during depositing. The tremie may be filled by one of the following methods: (1) Place the lower end in a box partly filled with concrete, so as to seal the bottom, then lower into position; (2) plug the tremie with cloth sacks or other material, which will be forced down as the pipe is filled with concrete; (3) plug the end of the tremie with cloth sacks filled with concrete. The concrete shall be discharged and spread by raising the tremie in such a manner as to maintain as nearly as practicable a uniform flow and avoid dropping the concrete through water. If the charge is lost during depositing the tremie shall be withdrawn and refilled.

(*b*) *Drop-Bottom Bucket.*—The bucket shall be of a type that cannot be dumped until it rests on the surface upon which the concrete is to be deposited. The bottom doors when tripped shall open freely downward and outward. The top of the bucket shall be open. The bucket shall be completely filled, and slowly lowered to avoid back-wash. When discharged, the bucket shall be withdrawn slowly until well above the concrete.

(*c*) *Bags.*—Bags of jute or other coarse cloth shall be filled about two-thirds full of concrete and carefully placed by hand in a header-and-stretcher system so that the whole mass is interlocked.

52. Great care shall be exercised to disturb the concrete as little as possible when it is being deposited in order to avoid the formation of laitance. On completing a section of concrete, the laitance shall be entirely removed before work is resumed.

FORMS.

53. Forms shall conform to the shape, lines and dimensions of the concrete as called for on the plans. Timber used in forms for exposed surfaces shall be dressed to a uniform thickness, and shall be free from loose knots or other defects. Joints in forms shall be horizontal or vertical. For unexposed surfaces and rough work, undressed timber may be used. Timber once used in forms shall have nails withdrawn and surfaces to be in contact with concrete thoroughly cleaned before being used again.

54. Forms shall be substantial and sufficiently tight to prevent leakage of mortar; they shall be properly braced or

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tied together so as to maintain position and shape. If adequate foundation for shores cannot be secured, trussed supports shall be provided.

55. Bolts and rods shall preferably be used for internal ties; they shall be so arranged that when the forms are removed no metal shall be within 1 in. of any surface. Wire ties will be permitted only on light and unimportant work; they shall not be used through surfaces where discoloration would be objectionable. Shores supporting successive stories shall be placed directly over those below, or so designed that the load will be transmitted directly to them. Forms shall be set to line and grade and so constructed and fastened as to produce true lines. Special care shall be used to prevent bulging.

56. Unless otherwise specified, suitable mouldings or bevels shall be placed in the angles of forms to round or bevel the edges of the concrete.

57. The inside of forms shall be coated with non-staining mineral oil or other approved material or thoroughly wetted (except in freezing weather). Where oil is used it shall be applied before the reinforcement is placed.

58. Temporary openings shall be provided at the base of column and wall forms and at other points where necessary to facilitate cleaning and inspection immediately before depositing concrete.

59. Forms shall not be disturbed until the concrete has adequately hardened. Shoring shall not be removed until the member has acquired sufficient strength to safely support its weight and the load upon it. Members subject to additional loads during construction shall be adequately shored to support both the member and construction loads in such a manner as will protect the member from damage by the loads; this shoring shall not be removed until the member has acquired sufficient strength safely to support its weight and the load upon it.

DETAILS OF CONSTRUCTION.

Reinforcement.

60. Metal reinforcement, before being positioned, shall be thoroughly cleaned of mill and rust scale and of coatings that will destroy or reduce the bond. Reinforcement appreciably reduced in section shall be rejected. Where there is delay in depositing concrete, reinforcement shall be re-inspected and, when necessary, cleaned.

61. Reinforcement shall be carefully formed to the dimensions indicated on the plans. Cold bends shall be made around a pin having a diameter of four or more times the least dimension of the reinforcement bars for steel of structural grade and eight or more times that for steel of intermediate or hard grade.

62. Metal reinforcement shall not be bent or straightened in a manner that will injure the material. Bars with kinks or bends not shown on the plans shall not be used. Heating of reinforcement will be permitted only when the entire operation is approved.

63. Metal reinforcement shall be accurately positioned, and secured against displacement by using annealed iron wire of not less than No. 18 gauge or suitable clips at intersections, and shall be supported by concrete or metal chairs or spacers, or metal hangers. The minimum clear distance between parallel bars shall be $1\frac{1}{2}$ times the diameter of round bars or $1\frac{1}{2}$ times the diagonal of square bars; if the ends of bars are anchored as specified in Section 140, the clear spacing may be made equal to the diameter of round bars or to the diagonal of square bars, but in no case shall the spacing between bars be less than 1 in., nor less than $1\frac{1}{4}$ times the maximum size of the coarse aggregate. Bars parallel to the face of any member shall be embedded a clear distance of not less than one diameter from the face.

64. In slabs, beams, and girders, splices of reinforcement shall not be made at points of maximum stress without approval. Splices, where permitted, shall provide sufficient lap to transfer the stress between bars by bond and shear. In such splices the bars shall be placed at the minimum distance specified in Section 63; adjacent bars shall not be spliced at the same point. Splices in columns, piers and struts shall provide sufficient lap to transfer the stress by bond.

65. Where changes in the cross-section of a compression member occur the longitudinal reinforcement bars shall be sloped for the full length of the member or offset in a region where lateral support is afforded. Where offset, the slope of the inclined portion from the axis of the member shall not be more than 1 in 6.

66. Exposed reinforcement bars in-

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tended for bonding with future extensions shall be protected from corrosion.

Protective Concrete Covering.

67. Metal reinforcement in wall footings and column footings shall have a minimum covering of 3 in. of concrete. At surfaces of concrete exposed to the weather, metal reinforcement shall be protected by not less than 2 in. of concrete.

68. Metal reinforcement in fire-resistive construction shall be protected by not less than 1 in. of concrete in slabs and walls, and not less than 2 in. in beams, girders and columns, provided aggregate showing an expansion not materially greater than that of limestone or trap rock is used; when impracticable to obtain aggregate of this grade, the protective covering shall be 1 in. thicker and shall be reinforced with metal mesh having openings not exceeding 3 in., placed 1 in. from the finished surface.

In structures where the fire hazard is limited, the metal reinforcement shall not be placed nearer the exposed surface than ¾ in. in slabs and walls or 1½ in. in beams, girders and columns.

Joints.

69. Joints not indicated on the plans shall be so designed and located as least to impair the strength and appearance of the structure. To prevent laitance in horizontal joints, excess water shall be removed from the surface forming the joint after depositing the concrete. Surfaces of contact shall be cleaned and wetted before depositing is resumed, and any laitance shall be removed. Where additional resistance to horizontal shear is required, stones shall be partially embedded in such a manner as to key with the adjoining concrete; or mortices or keys shall be formed in the concrete.

70. Joints in columns shall be made at the underside of the floor. Haunches and column capitals shall be considered as part of and to act continuous with the floor. At least two hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs.

71. Construction joints in floors shall be located near the middle of spans of slabs, beams, or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. Adequate provision shall be made for shear by use of inclined reinforcement.

72. Construction joints made crosswise of a building 100 ft. or more in length, shall have special reinforcement placed at right angles to the joint and extending a sufficient distance on each side of the joint to develop the strength of the reinforcement by bond. This reinforcement shall be placed near the opposite face of the member from the main tension reinforcement; the cross-sectional area of such reinforcement shall be not less than 0·5 per cent. of the section of the members cut by the joint.

73. Expansion joints shall be so detailed that the necessary movement may occur with the minimum resistance at the joint. The structure adjacent to the joint shall preferably be supported on separate columns or walls. Reinforcement shall not extend across an expansion joint; the break between the two sections shall be complete. A coating of white lead and oil, asphalt paint, petrolatum, or waterproofed building paper, placed over the entire surface of the hardened concrete, is commonly used for this purpose. Exposed edges of expansion joints in walls or abutments shall be rounded. Exposed expansion joints between two distinct concrete members shall be filled with an elastic joint filler of approved quality.

74. Buildings exceeding 200 ft. in length and of width less than about one-half the length shall be divided by means of expansion joints located near the middle but not more than 200 ft. apart, to minimise the destructive effects of temperature changes and shrinkage. Where there is an abrupt change in the width of a building an expansion joint shall be provided.

75. The seat of sliding joints shall be finished to a smooth plane surface and allowed to harden. Two thicknesses of building paper shall be placed on the seat before depositing superimposed concrete. Sheet zinc, lead, and bronze are also used for this purpose.

76. Where construction joints are required to be water-tight the method of construction shall be as follows:

(*a*) Horizontal joints shall be constructed by forming a continuous keyway in the lower portion of concrete

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before the concrete has hardened. Before placing the superimposed concrete the joint shall be thoroughly cleaned of laitance or other foreign material, saturated with water and coated with neat cement grout. The superimposed concrete shall be placed in such a manner as will ensure an excess of mortar over the entire surface of the joint.

(*b*) Vertical joints shall be made by a metal water-stop.

Seepage water shall be offered and drained from the forms, where required, vent pipes shall be closed after the concrete has thoroughly hardened. To secure water-tight joints it is vitally essential that all incoming water be drained from behind the forms during the process of concreting. If the upper portion of the joint is concreted first, a metal water-stop should be provided.

WATERPROOFING AND PROTECTIVE TREATMENT.

Waterproofing.

77. Concrete required to be water-tight shall be made with strict adherence to all provisions in these specifications regarding the choice of materials, proportions, consistency, mixing, placing, protecting, and workmanship.

78. Integral compounds shall not be used for waterproofing unless specifically authorised.

Oilproofing.

80. Concrete containers for light mineral oils, animal oils, certain vegetable oils and other commercial liquids shall be given an inside coating which shall be applied before the container is placed in service. Floors or other surfaces exposed to heavy concentrations of such oils or liquids shall be similarly protected. Concrete containers for mineral oils of 30° Baumé and lighter should be given a coating which will not be affected or penetrated by mineral spirits. Coatings containing ingredients which saponify or oxidise in the presence of lime shall be applied only to a surface which has previously been treated to neutralise the lime. Concrete containers for mineral oils heavier than 30° Baumé require no coating. Certain vegetable and animal oils and strong acids and alkaline solutions have a destructive effect on concrete and some protective treatment is required. Several vegetable oils have a disintegrating effect on concrete in cases where the surface is alternately wet and dry, yet when stored in closed concrete containers have no injurious action. Concrete containers for commercial liquids involve certain special features in addition to the general requirements of good concrete. Containers for such liquids often have thin sections, therefore a rich mixture is necessary to obtain concrete of proper workability without sacrificing strength and impermeability; it is especially necessary to eliminate joints or seams; thorough spading is essential and should be continuous during the placing of concrete.

Concrete in Sea Water.

81. Plain concrete in sea water from 2 ft. below low water to 2 ft. above high water, or from a plane below to a plane above wave action, shall contain a minimum of 7 cu. ft. of Portland cement per cubic yard in place. Other plain concrete in sea water or exposed directly along the sea coast shall contain a minimum of 6 cu. ft. of Portland cement per cubic yard in place. Porous or weak aggregates shall not be used.

82. The consistency shall meet the requirements of Section 30.

83. Sea water shall not be allowed to come in contact with the concrete until it has hardened for at least four days. Concrete shall be placed in such a manner as to minimise the number of horizontal or inclined seams or work planes. The placing of concrete between tides shall be a continuous operation, in accordance with Section 43; where it is impossible to avoid seams or joints, proceed as in Section 44. Concrete shall be deposited in sea water only when so directed by the engineer, in which case it shall be placed in accordance with the methods described in Sections 47 to 52.

84. Metal reinforcement shall be placed at least 3 in. from any plane or curved surface, except at corners when it shall be at least 4 in. from adjacent surfaces. Metal chairs, supports, or ties shall not extend to the surface of the concrete. Where unusually severe conditions of abrasion are anticipated the face of the concrete from 2 ft. below low water to 2 ft. above high water, or from a plane below to a plane above wave action, shall be protected by creosoted timber,

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dense vitrified shale brick, or stone of suitable quality.

Concrete in Alkali Soils or Waters.

85. Concrete in alkali waters or below ground-line of alkali soils shall contain a minimum of 7 cu. ft. of Portland cement per cubic yard in place. Under certain circumstances concrete is attacked by alkaline waters. The term "alkali" is here used to designate the soluble salts which occur in considerable quantities in the soils and waters of certain Western States; the sulphates, chlorides, and carbonates of sodium and magnesium are the most common forms of alkali. It is important to distinguish between the different forms of alkali which occur in nature. Experience and tests have shown that certain forms are more injurious than others; the chlorides and carbonates produce little or no injury to concrete. Special care must be used in placing concrete where it will be exposed to sulphate waters. An impermeable concrete made with a durable aggregate is necessary. Concrete should be permitted to harden under favourable conditions before it is exposed to injurious alkalies, and wherever practicable such concrete should be made in the form of pre-cast units.

86. The consistency of concrete in alkali soils or waters shall be such as to meet the requirements of Section 30.

87. Concrete shall be placed in such a manner as to minimise the number of horizontal or inclined seams, or work planes.

88. Metal reinforcement or other corrodible metal shall not be placed closer than 2 in. to the surface of members exposed to alkali soils or waters. In foundations and in heavy structures the metal reinforcement shall not be placed closer than 3 in. to the surface.

SURFACE FINISH.

89. The requirements in these specifications applying to forms and mixing, conveying, depositing, and finishing concrete, shall be followed unless modified by the plans.

The whole of a showing face between prescribed construction joints shall be cast in one continuous operation. Construction joints shall be true to line with sharp unbroken edges.

The same brand of cement and the same kind and size of aggregate shall be used throughout the whole of any showing face.

For showing faces the forms shall be smooth and water-tight. If wood be used the boards shall be planed, grooved and tongued, evenly matched, and tightly placed. They shall be so constructed as to be removable in sections by unscrewing or otherwise loosening them without hammering or prying against the face. Any offsets in the forms that may occur shall be smoothly dressed and any openings pointed flush with stiff clay or plaster of Paris in order to prevent leakage or the formation of fins.

Concrete that is to have a showing face, whether any particular finish is called for or not, shall be mixed, placed, and worked to secure at the face a uniform distribution of the aggregates, freedom from void spaces, and uniform texture. If the finish is required to be one that will expose the coarse aggregate by either scrubbing, tooling, sand-blasting, or acid treatment, then after the full surface of mortar has been worked against the form the spading tool shall be inserted in the concrete and the coarse aggregate pressed against the form in order to secure uniform distribution at the face and a uniform texture after the aggregate is exposed.

Face forms shall be removed as soon as practicable in order to facilitate effective repair of void spaces or broken corners before the surface has dried. Care shall be taken to avoid roughening or injuring corners, and to keep edges sharp.

As soon as the face forms are removed any fins or other projections shall be carefully removed, offsets levelled, and any voids or damaged places shall immediately be saturated with water and filled with a mixture of the same composition as that used in the surface, and brought even with the surface by means of a wooden spatula or float. A steel trowel shall not be used to finish the surface. The face shall be finished free from streaks, discolorations or other imperfections. Plastering will not be permitted.

Where a surface of mortar is to be the basis of the finish the coarse aggregate shall be worked back from the form with a suitable tool so as to bring a full surface of mortar against the form, care being taken to prevent the formation of voids and air pockets.

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Whenever forms are removed from showing faces before the concrete has become hard and dry, the surface of the concrete shall be immediately wetted and kept wet for at least three days.

90. Granolithic surfaces shall be made by placing about 1 in. of facing concrete against the face form in advance of the concrete, of such consistency and in such a manner as will ensure its bonding with the concrete.

The facing concrete shall be composed of 1 part Portland cement, 1½ parts fine aggregate, and 2½ parts coarse aggregate made up of pebbles, crushed granite, or other stone as called for.

If iron or wooden moulds are used to retain the facing against the forms while placing concrete, care should be taken that the mould is not permitted to remain until initial setting occurs. The moulds shall be jarred frequently and raised at short intervals to prevent formation of seams and air spaces between the surface and the concrete.

91. Top surfaces not subject to wear shall be smoothed with a wood float and be kept wet for at least seven days. Care shall be taken to avoid an excess of water in the concrete, and to drain or otherwise promptly remove any water that comes to the surface. Dry cement, or a dry mixture of cement and sand, shall not be sprinkled directly on the surface.

Wearing Surfaces.

92. Aggregates for the wearing surface shall have a high resistance to abrasion, and shall be screened and when necessary thoroughly washed. The least quantity of mixing water that will produce a dense concrete shall be used. The mix shall not be leaner than 1 part of Portland cement and 2½ parts of aggregate. The surface shall be screeded even and finished with a wood float. Excess water shall be promptly drained or otherwise removed. Overtrowelling shall be avoided.

93. The wearing surface in two-course work shall be placed within ½ hour after the base-course. Where the wearing surface is required to be applied to a hardened base-course the latter shall be prepared by roughening with a pick or other effective tool. The roughened surface shall be thoroughly saturated with water and covered with a thin layer of neat cement paste immediately before the wearing surface is placed. The wearing course shall not be thinner than 1 in.

94. Concrete wearing surfaces made in accordance with Sections 92 and 93, shall be kept wet for at least 10 days in the case of floors and 21 days in the case of roads and pavements. Prevention of premature drying during the early hardening of concrete is essential to the development of high resistance to abrasion. The surface may be covered with a layer of burlap, earth or sand, kept wet, or it may be divided into small areas by dikes and flooded with water to a depth of 2 or 3 in.

95 (*a*). Terrazzo finish shall be made by mixing 1 part of cement, 2½ parts of crushed marble or other stone, or crushed pebbles, and sufficient water to produce a dense concrete. The concrete shall be spread on the base-course and worked down to a thickness of 1 in. by patting or rolling and trowelling. The marble shall all pass a ½-in. screen and be free from dust. The surface shall be kept wet for not less than ten days, and after curing shall be rubbed to a plane surface with a stone or a surfacing machine. Hardened concrete to which a terrazzo finish is to be applied shall be prepared as prescribed in Section 93.

95 (*b*). Terrazzo finish shall be made by mixing 1 part of cement, 2 parts of sand, and sufficient water to produce a plastic mortar, which shall be spread on the base course to a depth of 1 in. Crushed marble, free from dust and passing a ¼-in. screen, shall be sprinkled over the surface of the fresh mortar and pressed or rolled in. The surface shall be kept wet for not less than ten days, and after curing shall be rubbed to a plane surface with a stone or a surfacing machine. Hardened concrete to wh ch a terrazzo finish is to be applied shall be prepared as prescribed in Section 93.

Decorative Finishes.

96. Immediately after the forms are removed the surface shall be wetted and rubbed with a carborundum brick or other abrasive until even and smooth and of uniform appearance, without applying any cement or other coating.

97. The forms shall be removed and the scrubbing done before the concrete has hardened. In warm weather this will require from 6 to 24 hours, and in cold weather from 1 to 3 days. The sur-

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face shall be scrubbed with fibre or wire brushes using water freely, until the surface film of mortar is removed and the aggregate uniformly exposed ; then rinsed with clean water. If portions of the surface have become too hard to scrub in equal relief, dilute hydrochloric acid (1 part acid to 4 parts water) may be used to facilitate the scrubbing. The remaining acid shall be thoroughly removed with clean water.

98. The concrete face shall be permitted to attain an intermediate degree of hardness ; it shall then be air-blasted with hard sand until the aggregate is in uniform relief.

99. The surface shall be permitted to become dry and hard, and then dressed with tools, as called for, by axing, rough or fine pointing, or bush-hammering to a uniform texture and even face.

100. The forms shall be removed before the surface has fully hardened ; the surface shall be rubbed with a wooden float by a uniform circular motion, fine sand being rubbed into the surface until the resulting finish is even and uniform.

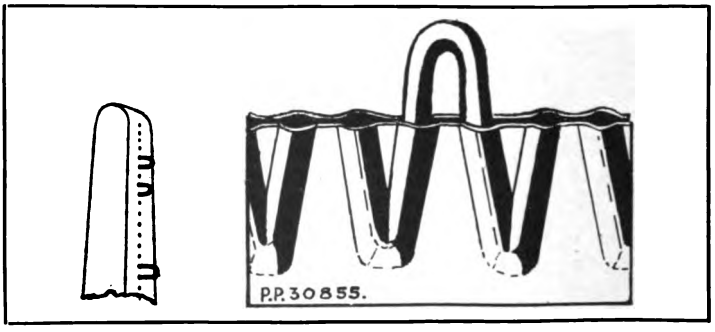
101. Coloured or other special aggregate used for finish shall be exposed by scrubbing as provided in Section 97. Facing mortar, made from this special aggregate, of 1 part of cement, 1½ parts of sand, and 3 parts of pebbles or screenings, clean and free from dust, shall be placed against the face forms to a thickness of about 1 in., sufficiently in advance of the concrete to prevent the latter coming in contact with the form.

102. Mineral pigment shall be thoroughly mixed dry with the cement and fine aggregate ; care shall be taken to secure a uniform tint throughout.

(*To be concluded.*)

Staple-Holder for Concrete Posts.

WITH the object of overcoming one of the difficulties experienced with concrete fence posts, namely, fixing wires at different heights from those at which holes or staples are cast, the Darlington Fencing Co., Ltd., of 5, Lloyds Avenue, E.C.3, have produced a simple device known as the " Darfen " staple-holder, illustrated herewith. As will be seen, it consists of strips of sheet steel which are cast in the post as it is made, leaving holes throughout the height of the post which permit staples to be driven in any



desired position. The strips are made in lengths of 16 in., so that three are required for a 4-ft. post, and from the price list seem to be quite cheap ; it would, in fact, probably be found that the cost of these staple-holders would be cheaper than inserting iron blocking pieces to form holes in the post, with the consequent labour of oiling and withdrawing them. It would, of course, be necessary to prevent the holes in the staple-holder being filled with concrete when the posts are cast, but this could be easily done by filling the openings with clay or similar material. The strips are made to take 1½-in. staples of 8-gauge galvanised wire with diamond-cut points, and as these are splayed when driven into the holder it is practically impossible to remove them except by cutting.

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Design of Formwork for Reinforced Concrete Construction.

By A. E. Wynn, B.Sc., A.M.Am.Soc.C.E.

XI.—MISCELLANEOUS FORMS IN BUILDING CONSTRUCTION.

Stair Forms.

STAIRS are designed to be self-supporting longitudinally from floor to floor or floor to landing, or across from wall to wall. The former method is more common when the whole frame of the building is concrete, and the latter when the floors only are concrete supported by brick bearing-walls.

The stairs may be a straight run from floor to floor (*Fig.* 64), but more often there is an intermediate landing (*Fig.* 64, dotted lines). Usually the stairs are not poured until after all the floors are completed, making this a separate operation from ground floor to roof. This saves time in pouring the main floors.

With straight runs from floor to floor ledges and keys are left in the floor beams to support the stair slab.

In order that the risers in each flight may come vertically over each other it is best to leave the stair opening a few inches wider than actually required to allow for adjustment horizontally; this clearance is shown as 6 in. in *Fig.* 64, 3 in. top and bottom. If the opening is made the exact size it is difficult to carry the edges up plumb throughout a high building.

The sloping slab is supported on 4 in. by 4 in. joists and ledgers. The posts should preferably be as nearly at right-angles to the ledgers as possible, with wedges between, but may be placed vertical if cleated to the ledgers and well cross-braced.

The side forms, or stringers, are cut out of 2-in. plank from 10 in. to 12 in. deep, as required. To these are nailed the riser forms, also of 2-in. plank.

On straight runs the stringers are cut as shown in *Fig.* 66, the dotted lines indicating the full plank and the solid lines the cuts. Starting from one upper corner of the plank, the length of the tread and riser is marked out successively, the top of each riser being on the edge of the plank. The clearance allowed—this should be measured from the actual opening—is then marked off and the plank sawn through parallel to the risers.

When there is a landing the clearance will only occur at the floor line, and at the landing the stringer ends at the junction of the stair and landing slabs, and a plank the depth of the landing slab is cleated on. The stringers are set in position on the sheathing, and may be nailed in place or held by a 1-in. ribbon.

The whole form is wedged up to the correct height. When there is a landing there should be a joist at the junction of the slabs.

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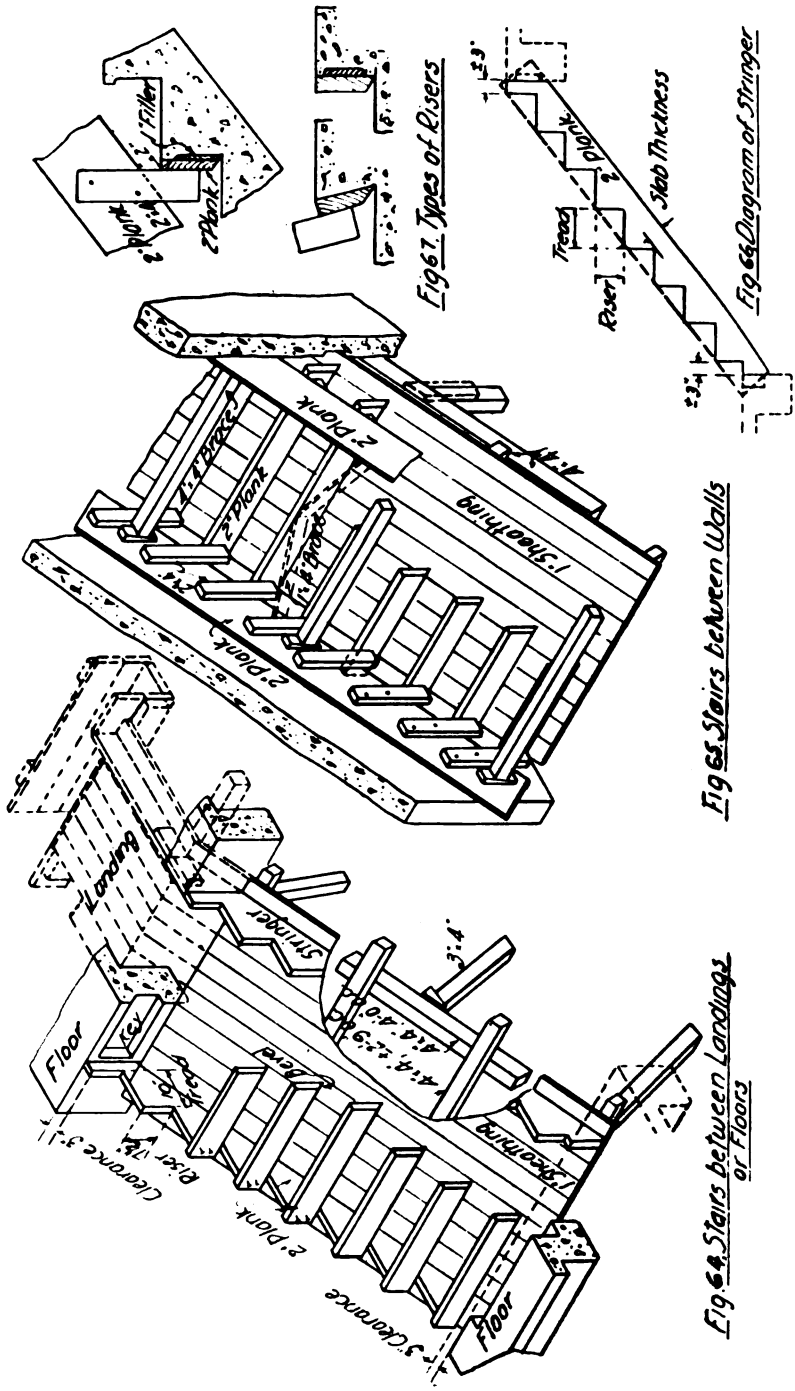

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The landing sheathing is carried on the beam side with intermediate joists as required by the span and thickness of the landing. The landing beam is usually poured with the stairs, being supported either by brackets built on the columns for that purpose, by independent columns, or by hangers from the beam above at the floor.

On the face of each stringer is nailed a 2-in. plank to form the riser, and of the same depth as the riser. The bottom of these planks should be bevelled so that the finisher can get his trowel into the corner, otherwise there will be a ridge left on the concrete. If each tread has a slight slope for drainage, this slope must be allowed for in laying out the stringer.

The risers may slope inwards, may be recessed, or have nosings, as shown in *Fig.* 67. To form these nosings a 1-in. filler-piece is nailed to the riser plank.

If the stairs are wide and there is danger of the riser planks bulging, a long plank or 2 in. by 4 in. can be placed on edge across the top of the risers nailed to posts wedged between the floors. For very wide stairs one or more intermediate stringers can be used inverted over the riser planks and held in the same way, or the planks can be wired back to the under form.

When the stairs are supported between walls already built (*Fig.* 65) the slab form will be the same as before, or if built on a fill no form is necessary. Sloping ledges are left in the wall to give bearing to the slab.

The riser forms are supported differently, as they must be hung from above. Two 2-in. planks are placed along the walls so that they will clear the risers by 2 in. or 3 in. They are braced together by 4 in. by 4 in. and wedges, or they can be supported by 4 in. by 4 in. posts at each end wedged between floors, or the planks can be bolted to the walls.

To the planks are nailed 2 in. by 4 in. vertical hangers at distances apart equal to the width of tread. The hangers should stop 2 in. or 3 in. above the treads. To the hangers are nailed the riser planks, with fillers attached when required. The risers must be cut the exact length between walls.

Stripping the risers can be done in one to two days, but it is advisable to leave them longer to protect the edges.

Cost.—The cost of stair forms can be estimated by the flight, by the square foot of under surface, or by the lineal foot of riser; the two latter methods give about the same result since there is usually about 1 sq. ft. of surface to 1 lineal foot of riser. An average flight from floor to landing will need 40 to 50 sq. ft. of forms, measured along the slope, for a 12 ft. story height and 4 ft. to 5 ft. width of stairs.

If the under-form is supported, each square foot will require about $\frac{1}{2}$ cu. ft. of timber, including stringers and risers. If the stairs or steps are built on a fill, no under-form being required, about $\frac{1}{3}$ cu. ft. per sq. ft. should be allowed.

The labour cost is best estimated by the flight from floor to landing. An average flight, as above, will be a day's work for two carpenters assisted by about $\frac{3}{4}$-hour labourers' time to each hour carpenters' time, including erection and stripping.

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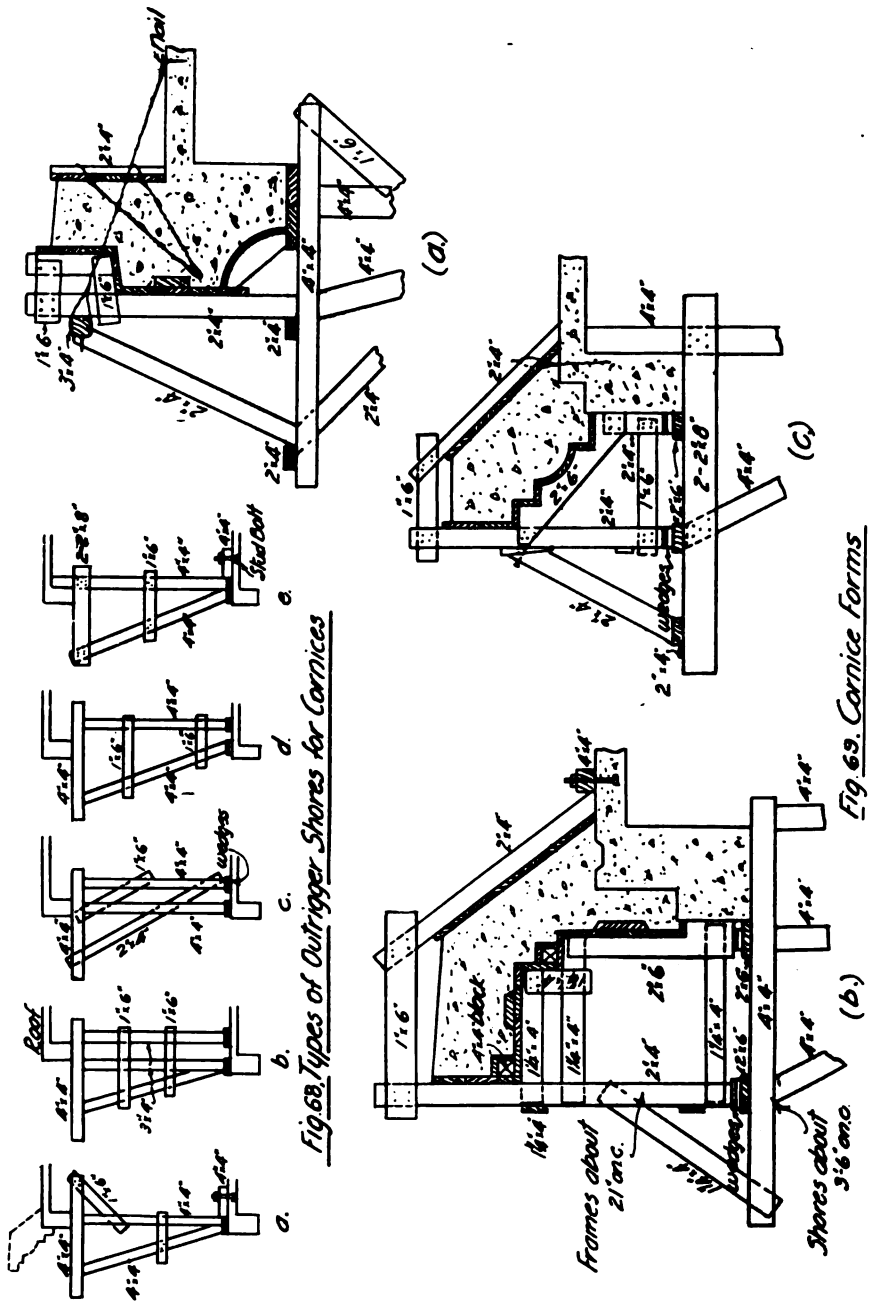

Fig. 68. Types of Outrigger Shores for Cornices

Fig. 69. Cornice Forms

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Cost of 1 flight of 40 sq. ft.—

16 hrs. carpenter @ ..... =
12 hrs. labourer @ ..... =
20 cu. ft. timber @ ..... =

The timber can be used several times over, so that the above amount can be divided by the number of times used.

For longer or wider flights the cost can be estimated proportionally by reducing to a square foot basis.

Cornice Forms.

Cornices are of so many different designs and shapes that it is only possible to describe the main points in the construction of the forms, the actual details being different with each design. If the cornice is heavy and complicated it is always built separate from the roof slab, the wall beams being keyed to receive it. Only the simplest of designs is built with the roof slab, because if it is at all complicated the time required to make and place the forms would hold up the pouring of the roof ; and also, since the forms are expensive to make, it is usual to make up a minimum length and use it over several times.

Generally the wall beams are stripped and reshored, but the outside and bottom forms may be left on if they are of any assistance in supporting the cornice form.

The method of shoring the cornice is very important, as most of the weight will overhang the building. They should be placed closer together than required for strength, generally from 3 ft. to 4 ft. As the posts cannot be placed vertically under the load there will always be some bending action, so that at least two posts should be used and well cross-braced together. The shores will be what are known as "outrigger shores," the outside arm being longer than the inside arm. They consist of two or more posts, cap and braces.

Various types of outrigger shores are shown in *Fig.* 68. The type to use will depend mainly on the design of the cornice. If the cornice is vertical (that is, if most of the weight comes on the roof), type (*a*), (*c*) or (*d*) can be used. If the cornice overhangs, types (*b*) and (*e*) would be better. The main points to remember are that the posts will tend to be pulled outwards at the top and pushed inwards at the bottom. When a vertical and inclined post butts at the bottom, as at (*a*) and (*e*), kick blocks should be bolted to the floor to take the thrust. At (*e*) is shown a good form of shore, where the inside post is wedged against the roof slab, and the cap consists of two planks nailed to the sides of the posts. Batter posts should be under the point of application of the load, so that there is little bending in the cap. All posts should be on wedges.

On top of the shores are set the built-up frames to which is nailed 1 in. or 1¼ in. sheathing. The type of frame will vary with the design of the cornice, but is generally made up of 1¼ in. or 2 in. by 4 in. nailed together and braced. At (*a*), (*b*) and (*c*), *Fig.* 69, are shown typical designs. When there is only one vertical leg to the frame, as at (*a*), it

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should be braced from the cap and tied back to the roof. In this design the curved portion is a separate unit lightly nailed to the side and bottom. If the form is to be stripped quickly the piece forming the inset should be nailed on lightly so that it will remain in the concrete and protect the edges in stripping. A 3-in. by 4-in. wale should be used to hold in line the uprights, and when there is any pressure on the inclined brace to the wale the end of the cap should be braced back to the posts.

When the frames are wide, as at (*b*) and (*c*), wales need not be used, but diagonal braces are advisable to prevent any tendency of the form to be pushed outwards. In designs such as (*b*) and (*c*) it is best to set the frames on wedges so that they can be lowered to clear the concrete before swinging them out ; this will prevent corners being broken off.

When there are undercuts or recesses in vertical faces, as in (*b*), the boards to form these should be nailed on lightly to the form, so that they will remain in the concrete when stripping. If the nailing is done from the outside, leaving the heads projecting, the nails can be withdrawn leaving the inset board loose to enable the form to be lowered.

At (*b*) the frame is built to take care of the main details of the cornice, and the remaining details are blocked out with independent forms of sheathing nailed to 4 in. by 4 in. blocks. This method should be used whenever possible, as it makes a strong form and there is no expensive labour cutting members to the exact outline of the cornice.

At (*c*) the template method is used, the 2 in. by 6 in. joists being cut to the outline.

Frames should be 18 in. to 24 in. apart, depending on the weight to be carried. There should be shores at alternate frames. Intermediate frames are supported by 2-in. continuous planks, unless the pressure is mostly horizontal, as at (*a*), when no support is necessary. Some longitudinal bracing is necessary to add stiffness to the form.

The complete form is usually built up on the ground or on the roof in lengths of 6 ft. to 10 ft., depending on the weight. A light hand-derrick is necessary for placing the forms and handling them while stripping. The back-form of the cornice can be held in several ways, as shown in *Fig.* 69. Wires anchored around the reinforcing steel in the concrete are useful for this purpose.

The length of time the forms should be left before stripping depends on the overhanging weight. Type (*a*) could be stripped in two to three days, while (*b*) and (*c*) should be left a little longer. When the forms are to be used several times a short length can be stripped and immediately reshored from the main shores.

When there is a moulded belt-course running around the building the forms for it can be built in the same way as for cornices. If it is simple it will be cast with the floor, but if elaborate will usually be built afterwards, keys being left in the concrete to support it.

Cost.—Cornice and belt-course forms can only be estimated by experience, since they vary so much in design, but they will cost from two to four times as much as ordinary beam forms per sq. ft. of contact area.

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Sill Forms.

Concrete sills are poured after the walls are built and the steel window frames set, as they hold the bottom of the frames. The form (*Fig.* 70) is built in two halves, each consisting of a plank with 2 in. by 4 in. uprights attached to form a clamp. A 2 in. by 4 in. cross-piece nailed on to the top of the uprights holds them the right distance apart and a bolt draws them tight against the wall, with spreaders to give a uniform width of sill. The outside plank will have a strip nailed on the bottom to give the overhang, and to this strip is nailed the drip-moulding, which may be triangular or half round. This drip-moulding should only be nailed on very lightly, as it should stay in the concrete until it can be taken out without breaking off the outside edge of the sill. The sills can be stripped within two days.

Cost.—The unit cost of sills is usually estimated by the lineal foot of sill, which is about equivalent to 1 sq. ft. of contact area. A lineal foot will require about $\frac{1}{3}$ cu. ft. of timber, but the timber can always be used several times. The labour required to place and strip will be about 10 minutes carpenters' time plus 10 minutes labourers' time per lineal foot of sill.

Upturned Beams.

Exterior wall beams are often turned up to give more light to the building (*Fig.* 71). The beam and slab should be poured together, so the inside form of the beam must be supported above the slab. The side-form is built in the ordinary way, but a 2-in. plank is nailed on the bottom ; otherwise the concrete will come up on the inside and make it difficult to strip. This plank must be held down against upward pressure, and this can be done with wires anchored to the reinforcing steel with wedges to take up the slack. To support the form above the slab, temporary blocks of wood can be used and knocked out during concreting, or concrete blocks can be left in the slab. The outer end of the joists can be supported on two 2 in. by 6 in., separated as shown, using short intermediate joists if necessary to cut down the span of the sheathing carrying the extra weight of the beam.

Cost.—These beams will cost more than interior beams, and an allowance of about 25 per cent. more per sq. ft. can be made.

Column Mouldings and Ornaments.

Columns are sometimes marked out to make the concrete resemble cut stone work by nailing to the main form narrow strips of mouldings of the desired shape (*Fig.* 72). These strips should always be bevelled for easy stripping. Nailing and mitering the corners of the strips must be done very carefully for satisfactory results. The strips should preferably be nailed on lightly so that they will remain in the concrete, otherwise some edges are sure to break off if the strips and forms are removed together while the concrete is still green, and early stripping is desirable so that the concrete can be finished. With very high columns the strips,

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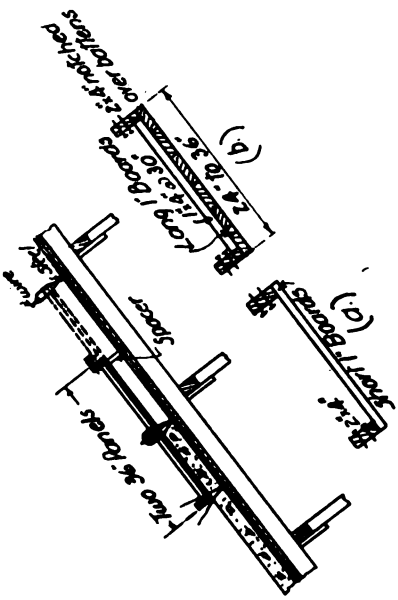


Fig. 73. Backforms for Sloping Slabs

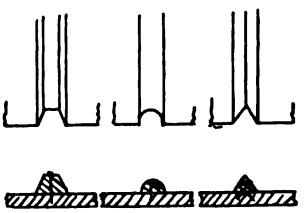


Fig. 72. Column Marking

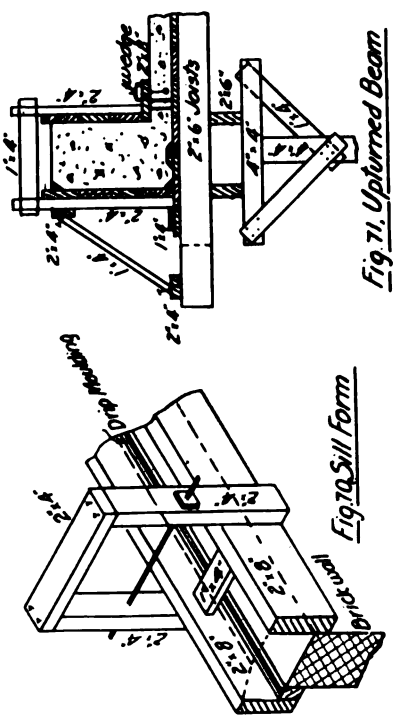
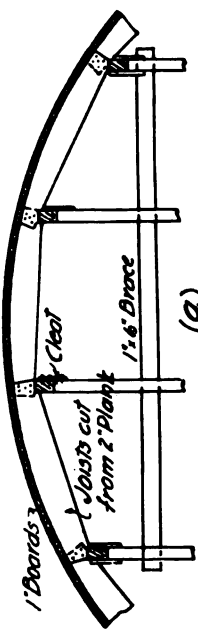


Fig. 71. Upturned Beam

Fig. 70. Sill Form



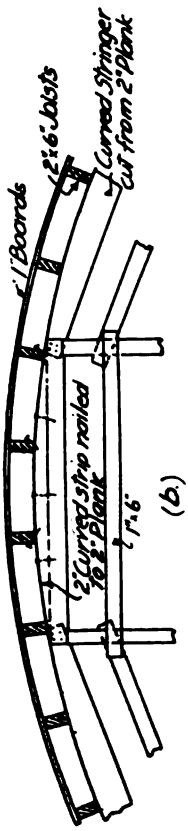


Fig. 74. Curved Slab Forms

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if nailed on lightly, may become dislodged during concreting, so in this case should be more securely nailed and the forms left on longer. The forms should be well oiled.

Inset ornaments and slots are made by cutting boards to the desired shape and nailing them to the main form.

Projecting ornaments, especially if they are elaborate, are generally more easily handled by precasting and setting the finished moulding in recesses in the concrete or by setting in the forms and pouring around them.

Cost.—The cost of ornamental work can vary enormously, and can only be estimated by experience. Any elaborate form should be made in a joiners' shop, and definite quotations can be obtained.

Back-forms.

When a concrete slab is poured on a slope, as, for instance, for a saw-tooth roof, a top or back-form (*Fig.* 73) is required to hold the upper surface of the concrete. It is hardly possible to pour concrete on a slope greater than thirty degrees with only an under-form, and even on this slope great care is required to prevent the concrete from running down the slope. It is not practical to build the form the full height before concreting, as it would be almost impossible to pour a thin slab from the top without getting numerous voids.

The form is best built in panels 2 ft. to 3 ft. wide and as long as the slab, or if this is too great in about 12 ft. lengths.

Using two panels, the first is placed and wired down to the reinforcing steel, using 1 in. sq. spacer blocks to hold it above the lower form ; then the form is filled to the top and the next panel is nailed on, wired down, and filled. The lower panel is then removed after drawing the nails and cutting the wires, moved ahead of the second one and so on. Using three panels the concrete is poured to the top of the third before removing the lowest one, allowing a little more time for the concrete to set.

In warm weather a panel can be removed in an hour, but the concrete should not be trowelled immediately as this would cause a tendency to flow.

The panels can be made most economically with the greatest salvage of material by running the boards lengthways in long lengths with 1 in. by 4 in. battens about 30 in. apart, and on each edge is nailed a 2 in. by 4 in. on edge, notched over the battens so that they can be nailed to the boards. These 2 in. by 4 in. act as wales and for nailing the panels together (*Fig.* 73 (*b*)). By this method the battens are the only short pieces required.

Another method (*Fig.* 73 (*a*)) is to run the boards the height of the panel with 2 in. by 4 in. on each edge and no battens, but this means cutting up the boards into short lengths with consequent waste.

The wires can be tied around the joists of the lower form, but this is more expensive as holes have to be bored ; and it is not necessary, as the pressure on the back-form is small. There should be a wire to about

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every 15 sq. ft. The back-forms should be oiled as well as the lower form, as they have to be stripped quickly.

Cost.—The cost of back-forms is about the same as the cost of making, placing and stripping slab-panel forms in beam and girder construction.

Curved Slab-forms.

Curved slab-forms (*Fig.* 74) can be built in two ways. Either the joists themselves can be cut to the required radius and supported by straight ledgers, or the joists can be straight and the ledgers cut to the radius. The first method would be better for a sharp curve and small area, and the latter for a flat curve and a large area.

At (*a*) the joists of 2-in. plank are cut as described for curved walls, nailed together, and notched at the joints to give square bearings for the ledgers. The posts should be well cross-braced to take the side thrust.

At (*b*) the joists are toe-nailed to the curved ledgers to hold them in place. The ledgers should be notched to give the posts square bearing, or the tops of the posts can be shaped, or wedges can be used. Instead of cutting the ledgers they can be left square and curved strips can be nailed on the upper edge. By this means there is full salvage of the ledgers and only the strips are wasted.

In power house construction there is often the problem of building forms for a slab that is flat at one end and curved at the other, the curvature gradually increasing from a straight line to a maximum curvature. This can be done by using straight joists resting on a horizontal ledger at one end and a curved ledger at the other. The joists are placed at right-angles to the curved ledger. The ends of the joists on the horizontal ledger must be bevelled to give a square bearing. The joists should preferably be heavy and so close that an intermediate ledger will not be required. If a board is laid over the joists it will bear only on one edge of each joist, so that the top of each joist should be bevelled to give full, or nearly full, bearing to the boards. Square-edge boards should be used for sheathing; as many boards as possible are nailed together, then a space is left afterwards to be filled with a special board, as described for sloping curved walls. The change in curvature in the width of a board will be so slight the small ridges formed in the concrete will hardly be noticeable.

Cost.—Curved slab-forms will require about $\frac{1}{3}$ cu. ft. of timber per sq. ft. of contact area, which can be divided by the number of times used. To erect and strip 100 sq. ft. of forms will require about 12 hours carpenters' time and 10 hours labourers' time.

(*To be continued.*)

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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Thames ballast	,,	10	6
Broken brick (¾ in.)	,,	11	6
Best British Portland Cement	per ton 58s. to	63	6
"Ferrocrete" Rapid-Hardening Portland Cement	delivered London 10s. per ton extra		
"Super-Cement"	per ton 88s. to	93	0
"Lightning" Brand Aluminous Cement	,, 108s. to	113	6
Ciment Fondu	130/- ex stores		

BOARDING FOR SHUTTERING—		Sawn. s.	Sawn. d.	Wrot. s.	Wrot. d.
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in.	from £23 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £23 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	11	0
9/16 in. to ½ in. Rounds	,,	11	3
⅜ in. Rounds	,,	11	6
¼ in. Rounds	,,	12	6

Breeze Slabs per yd. super: 2 in., 1/11; 2½ in., 2/4; 3 in., 2/9; 4 in., 3/6.

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 : 2 : 4—		s.	d.
Do. do. in foundation	per yard cube	48	6
Do. do. in columns	,, ,,	53	6
Do. do. in beams	,, ,,	53	6
Do. do. in floor slabs 4 in. thick	per yard super	5	8
Do. do. in floor slabs 6 in. thick	,, ,,	8	5
Do. do. in floor slabs 9 in. thick	,, ,,	12	3
Do. do. in walls 6 in. thick	,, ,,	8	8

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—		s.	d.
From ¼ in. to ⅝ in.	per cwt.	23	0
,, 9/16 in. to ½ in.	,,	22	0
,, ⅝ in. to 2⅞ in.	,,	21	0

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1¼*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2½*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.* (per bend per cwt.).

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.* (per bend per cwt.).

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	59	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	55	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11½
Do. do. as last, in narrow widths	,, ,,	1	1½
Do. do. to sides and soffits of beams, average 9 in. by 12 in.	,, ,,	1	1½
Do. do. as last, in narrow widths	,, ,,	1	3½
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	,, ,,	0	3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/9½ per hour; Carpenters working on old shuttering, 1/10½; Labourers on building works, 1/4½; Men on mixers and hoists, 1/5½; Bar-benders, 1/5½.

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Concrete at the Public Works, Roads and Transport Exhibition.

CONCRETE takes such an important part in public works of all kinds that, as was to have been expected, it occupied a prominent position at the Public Works, Roads and Transport Exhibition held at the Agricultural Hall last month.

The stand of the British Portland Cement Association was devoted entirely to the use which is being made of concrete by public authorities. In front of the stand was a complete section of reinforced concrete road, with two strips exposed to show the method of construction. The concrete for this road was made with rapid-hardening Portland cement, by the use of which a road can be opened to traffic within a week instead of the four or five weeks required when ordinary Portland cement is employed. At one side was a small section showing the method of forming the expansion joints; the edges of the concrete are rounded to a radius of ¼ inch, and the joint, ⅜ in. wide, filled with a strip of bituminous material which is allowed to stand up not less than ½ inch above the surface. This is afterwards beaten down flat and forms a strip about 1 inch in width, thus affording a complete protection for the joint; this method has been proved in America and in this country to be one of the most efficient forms of expansion joint.

Other exhibits on this stand were kerbs, channels and a pavement gutter, lent by the Aylesbury Borough Council, and a catch-pit, sectional culverts and a verge drain, lent by the Gloucestershire County Council. The last named drains the water from the road, under the grass verge, to the ditch, in order to avoid open "grips" which may he a danger to pedestrians. The Atlas Stone Co. supplied paving slabs and a set of kerbs, and the Premier Artificial Stone Co. supplied another length of kerbing.

Several excellent examples of cast or reconstructed stone were exhibited here, such as blocks resembling Ham Hill stone and columns representing red sandstone and Doulting stone.

An interesting exhibit was a panel illustrating the method adopted in the renovation of Witney Bridge, Oxfordshire. The stone of the old bridge was crushed and graded, and recast with Portland cement exactly to match the original. These exhibits were produced by the Wharf Lane Concrete Co. Two exhibits showing excellent finish were a four-armed direction post, as adopted by the Gloucestershire and other County Councils, lent by the Empire Stone Co., and a lamp post complete with lamp loaned by the Midland and Great Northern Railways Joint Committee.

Hard tennis courts are much in the public eye at the moment, and an example of the most recent form of construction was exhibited. This type of court is red in colour, and is so porous that when a bucket of water is poured on the surface the water entirely disappears in less than fifteen seconds, so that play may be resumed immediately after a shower of rain. The surface coat of ¾ in. consists of crushed red brick and cement, below which is a layer of clinker and cement, the whole lying on loose clinker or ashes. Other exhibits were multi-coloured concrete bricks, different types of fencing, etc.

Messrs. Tarmac, Ltd., appear to have increased the range of their concrete specialties, their exhibit this year including a garden roller, fence posts, kerbs, garden edging, and ornamental products such as moulded figures, vases and pedestals, ball-cap for gate piers, and a coat of arms. The stand of Messrs. Girling's Ferro-concrete Co. included permanent white-line blocks, cable covers, door hoods and brackets, ornamental gate piers, and concrete air bricks. The Croft Granite, Brick and Concrete Co. exhibited various concrete goods in which their granite is used as the aggregate, including sectional manholes, fence and rail posts, indicator posts, door hoods and brackets, paving flags, kerbs and channels. Similarly the Shap Granite Co. exhibited various concrete articles made from their own granite.

All the principal reinforcement firms were represented, but their systems are so well known as not to need a detailed description, but there were one or two new things to be seen among these exhibits On the stand of the Walker-Weston Co was a "Macrete" concrete road-surface, which is really a cement grouted surface,

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2 in. thick, which remains permanently rough. The British Reinforced Concrete Engineering Co. are now producing a double reinforcement; in the specimens exhibited, the bottom layer was twice the weight of the top, but this, of course, can be modified according to the conditions under which it is to be used.

On the concrete machinery stands there was not much that is new. Messrs. Winget showed a new hand-pressure machine in which the block is released with one movement only. The Ransome Machinery Co. exhibited a new block-making machine, which is claimed to be the only entirely automatic block-making machine in the world. This firm also showed a selection of their well-known mixers in various patterns and sizes, steel tip-carts, piling plant, etc. Messrs. Builders and Contractors Plant, Ltd., exhibited a new design of "Roll" mixer, specially adapted for road work; in a small and portable machine, the principal features of the larger machine, such as mixing by centrifugal force and gravity and quick discharge, have been retained. Washing, crushing and screening plant were also shown on this stand.

Messrs. Ruston & Hornsby, Ltd., showed a three-cylinder vertical oil engine, a pumping plant, boilers, horizontal oil engines of different types, and an excavator which may also be used as a dragline, grabbing crane, trench excavator, pile-driver, and crane. This plant may be mounted on rail wheels, road wheels, or caterpillars. As a crane navvy it has a jib 20 ft. long and a bucket of 7/9 cu. yd. capacity; as a dragline or a grabbing crane the jib is 30 ft. long and the bucket 7/9 cu. yd.; either a steam engine or electricity may be used for power.

Prospective New Concrete Work.

ABERAYRON. — *Bridges.*—The R.D.C. proposes to spend £370 on the construction of two bridges over the brook at Cribyn.

ABERDEEN.—*Houses.*—At a cost of £112,560 the Corporation Housing Committee recommends the erection of 268 houses.

ATCHAM.—*Bridge.*—The Salop C.C. proposes to build a bridge in place of the existing structure.

BADDINSGILL.—*Reservoir.*—The Bathgate D.C. has approached the West-lothian C.C. for the provision of a reservoir.

BEBINGTON.—*Bridge.*—The Cheshire C.C. has instructed the Surveyor to prepare plans for a bridge over the main road at Bebington.

BILLINGHAM. — *Houses.*—The U.D.C. has asked for a loan of £32,165 for housing purposes.

BIRSTALL.—*Road.*—The U.D.C. and West Riding C.C. are considering a plan for the construction of a road from Bradford Road to Church Road.

BLYTH.—*Promenade.*—The T.C. has asked the M.T. to contribute £17,000 for the extension to the promenade.

BRAINTREE.—*Houses.*—The U.D.C. has applied for sanction to erect 50 houses.

BRIGHTON.—*Houses.*—The C.B. is seeking permission to build 50 houses.

BRISTOL.—*Street Works.*—A sum of £11,000 is to be spent on concreting 25 cobbled streets.

BROADSTAIRS. — *Water Tank.* — The U.D.C. proposes to erect a 250,000-gallon water tank.

CAMBUSKENNETH.—*Bridge.*—The Stirling C.C. is considering a proposal to erect a bridge over the Forth between Stirling and Cambuskenneth. The cost is estimated at £12,862.

CHORLEY.—*Houses.*—The R.D.C. has applied to the M.H. for sanction to build 50 houses.

CORK.—*Concrete Bridge.*—The T.C. proposes to build a reinforced concrete bridge at Carrolls Dock, estimated to cost £2,500.

COWSEN.—*Bridge.*—The R.D.C. has instructed the Surveyor to prepare a scheme for the construction of a bridge at Cowsen.

CULLERCOATS.—*Houses.*—The Tynemouth Corporation is considering a scheme for the erection of 50 houses at Cullercoats.

DARLINGTON.—The T.C. has applied for a grant for the construction of a road and culvert from Drury Street to Hopetown Lane.

DULVERTON.—*Road.*—A scheme for the construction of a road from Coupleham Cross to Two Gates has been submitted to the M.T. The cost is estimated at £120,000.

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EARSDON.—*Houses.*—The U.D.C. proposes to build 300 houses, at an estimated cost of £397 each.

EASINGTON.—*Houses.*—The R.D.C. is taking up loans to the extent of £93,000 for housing purposes.

EMSCOTE. — *Houses.* — The Warwick T.C. is considering a scheme for the erection of 60 houses.

EPSOM.—*Reservoir.*—The U.D.C. is preparing a scheme for the construction of a reservoir of 2,000,000 gallons capacity.

FALLINGROYD. — *Bridge.*—The West Riding C.C. is considering a scheme for the construction of a bridge at Fallingroyd. The cost is estimated at £9,200.

FOLESHILL.—*Houses.*—The R.D.C. has applied for permission to erect 754 houses during the next two years.

HAMSTERLEY.—*Road.*—The Lanchester R.D.C. has applied for a grant for the construction of a road from Westwood Station to the main road at Hamsterley Colliery.

HASTINGS.—*Houses.*—The T.C. proposes to build 50 houses in the Red Lake district.

HEBDEN BRIDGE.—*Footbridge.* — The U.D.C. has a scheme in hand for the construction of a footbridge across the river at Harcastle Craggs.

HOUGALL.—*Concrete Jetties.*—The Durham C.C. proposes to construct eight reinforced concrete jetties at the river bank at Hougall Farm.

HULL.—*Bridge.*—The Corporation has applied for sanction to construct a bridge across the river, at an estimated cost of £275,000.

HULL.—*Wall.*—A concrete wall is to be built at the Ferens boating lake at a cost of £1,600.

ILKLEY.—*Houses.*—The U.D.C has applied to the M.H. for sanction to erect 42 houses in Leeds Road.

KILLARA. — *Reservoir.* — A sum of £300,000 is to be spent on the construction of a reservoir with a storage capacity of 30 million gallons.

KILLERMONT.—*Bridge.*—A sum of £4,000 is to be spent on the construction of a bridge across the River Kelvin at Killermont.

LEEDS.—*Roads.*—The Corporation has applied for assistance in connection with the construction of roads from Potternewton Lane to Meanwood Housing Estate; the junction of York and Selby Roads to Foundry Lane; and along the Wye Beck to the footpath leading to Low Coldcotes.

LEICESTER.—*Houses.*—The City Council is recommended to purchase Braunstone site for housing purposes. The cost is estimated at £115,000.

LEVEN.—*Houses.*—The Corporation Housing Committee is considering a proposal to erect 52 houses.

LINCOLN.—*Houses.*—Direct labour is to be used on the construction of 32 houses.

LOCHEE.—*Road.*—A road from Loons Road to Law is to be constructed at a cost of £9,000.

LONDON (ST. PANCRAS).—*Concrete Bridge.*—The B.C. proposes to erect a reinforced concrete bridge across the Regent's Canal at Kentish Town Road.

MANCHESTER.—*Houses.*—The Corporation Housing Committee are seeking permission to build 1,000 houses.

MILLOM. — *Road.* — The U.D.C. has approached the M.T. with a view to constructing a road from Furness to Whitehaven.

MITCHAM.—*Houses.*—The U.D.C. proposes to erect 210 houses on Swain's Marsh.

MORETONHAMPSTEAD. — *Reservoir.*—The Newton Abbot R.D.C. has instructed the Engineer to prepare a scheme for the construction of a 200,000-gallon reservoir for the parish of Moretonhampstead.

NEWQUAY.—*Promenade.*—The Newquay R.D.C. proposes to build a promenade on the sea front at Gowan.

RHYL.—*Bridge.*—A scheme is being considered for the erection of a bridge at Foryd Harbour to connect Flintshire with Denbighshire.

ROKER.—*Swimming Pool.*—The Sunderland Corporation proposes to spend £30,000 on the construction of a swimming pool on the rocks at Roker.

STARETON.—*Bridge.*—The Warwickshire C.C. proposes to erect a bridge in place of the present structure at Stareton.

SUNDERLAND.—*Quay.*—A proposal to spend £250,000 on the construction of a deep water quay is being considered.

SURBITON.—*Houses.*—The U.D.C. has applied for a loan of £26,000 for the erection of 50 houses.

TOTTENHAM.—The U.D.C. proposes to carry out improvements at Moselle Brook, at a cost of £1,600.

WALLASEY.—*Houses.*—The T.C. proposes to erect 270 houses on a site in Somerville Ward.

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Tenders Accepted.

ARDEE.—*Concrete Houses.*—The T.C. has accepted the tender of Messrs. McEnteggart Bros., of Nobber, for the construction of concrete houses at £227 each. Other tenders submitted were: J. Wynne, Dundalk, £390; J. McGuiness, £287 10*s.*; and D. Clarke, Ardee, £244.

BOURNEMOUTH.—*Retaining Walls.*—The T.C. has accepted the tender of Messrs. F. A. Grigg & Sons, Boscombe, at £895; of Messrs. Mark Loader & Sons, at £1,265 16*s.*; and of Messrs. Field & Cox, at £239, for the construction of concrete retaining walls.

CARLETT.—*Road.*—The Cheshire C.C. has accepted the tender of Mr. C. L. Warren, Seacombe, at £6,590, for the construction of a reinforced concrete road.

GLASGOW.—The T.C. has accepted the tender of Mr. W. T. Aikenhead, at £909 5*s.* 6*d.*, for reinforced concrete work in Richmond Park.

GRIMSBY.—*Concrete Floors.*—The T.C. has accepted the tender of Messrs. J. H. Thompson & Sons, at £8,612, for the construction of reinforced concrete floors at the building depot.

PORTSMOUTH.—The T.C. has accepted the tender of Messrs. F. Bevis, Ltd., at £1,795, for pulling down and removing 900 ft. of concrete seat and the construction of a reinforced concrete seat, including a flight of steps, in its place. Other tenders submitted were: J. Croad, £2,220; Mears Bros., £3,300; Malcolm Macleod & Co., Ltd., £4,800.

RAWTENSTALL.—*Flags.*—The T.C. has accepted the tender of the Shap Granite Co., Ltd., for the supply of 1,600 square yards of concrete flags.

ROYTON.—*Culvert.*—The U.D.C. has accepted the tender of Messrs. J. Spencer & Bros., at £1,700, for the construction of a concrete culvert at the sewage works.

SUDBURY.—*Bridge.*—The Middlesex C.C. has accepted the tender of Messrs. Peter Lind & Co., at £12,461 14*s.* 9*d.*, for the construction of a reinforced concrete bridge over the railway.

UXBRIDGE.—*Reservoir.*—The U.D.C. has accepted the tender of Messrs. Leslie & Co., Ltd., at £5,729, for the construction of a 1,000,000-gallon reservoir. Other tenders submitted were: A. E. Farr, £8,100; Industrial Constructions, Ltd., £7,560; T. Muirhead & Co., Ltd., £7,200; A. Jackaman & Son, Ltd., £6,962; Mears Bros., £6,881; Peter Lind & Co., £6,730; Anglo-Danish Concrete Construction Co., £6,483; Concrete Piling, Ltd., £6,271; K. Holst & Co., £5,985.

SITUATIONS VACANT.

CAPABLE ESTIMATOR, not over 45, experienced in calling on Architects, wanted by CONCRETE SPECIALISTS. State age, experience, and salary to Box 1257, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 20 Dartmouth Street, Westminster, S.W.1.

REINFORCED Concrete Designers with specialist experience required by a leading firm. Applications giving a summary of experience, age and salary required to be made in the first instance to Box No. 1258, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 20, Dartmouth Street, Westminster, S.W.1.

New Companies Registered.

BURTON HOLT (CHELSEA), LTD. (209066). Registered October 17. 1–3A, Church Street, Chelsea, S.W.3. Manufacturers of pre-cast concrete units and concrete *in situ*, etc. Nominal capital: £500 in £1 shares. Directors: C. V. Burton, 39, Clifton Park Avenue, Raynes Park; A. A. Holt, Clifton Hotel, Southend-on-Sea (managing director).

ENGINE FOUNDATION AND CONCRETE CONTRACTORS, LTD. (209242), 27, Soho Sq., W.C. Registered October 26. Builders in concrete, etc. Nominal capital: £5,500.

MONO CONCRETE CO., LTD. (209107). Registered October 19. Manufacturers of concrete pipes. Nominal capital: £8,000. Permanent directors: T. A. Womersley, 3, Elderslie Road, Eltham, S.E. (Chairman); B. Heffer, Cricket Field Cottage, Esher.

NOVOCRETE AND CEMENT PRODUCTS CO. (NORTHERN AREA), LTD., 3, Regent Street, S.W.1 (209200). Registered October 20. Nominal capital: £20,000.

REINFORCEMENTS, LTD. (209487). Registered November 5. 28, East Parade, Leeds. Reinforced concrete and structural steel engineers, etc. Nominal capital: £1,000. Directors: J. W. Liversedge, "Dunollie," The Avenue, Haton, near Leeds; Mrs. F. Liversedge and N. B. Liversedge.

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Recent Patent Applications.

216,892.—L. Rott: Doors and door-jambs of reinforced concrete.

217,605.—S. Seailles: Polishing concrete surfaces.

229,329.—F. Ranwald: Moulds.

235,138.—C. Pontoppidan and H. P. Boude: Manufacture of cement.

236,746.—Hume Pipe Co. (Australia), Ltd., and W. R. Hume: Concrete slabs.

237,347.—J. R. Wade: Reinforced-concrete structures and removable reinforcements therefor.

237,438.—G. W. Stokes: Falsework for construction of concrete floors.

237,515.—L. Sindic: Manufacture of cement.

237,779.—E. C. Eckel: Process of making cement and iron.

237,996.—E. G. Enticknap: Machine for manufacturing concrete blocks.

238,029.—R. W. De Montalk: Facing of concrete slabs and buildings.

238,331.—A. W. Dickinson: Reinforced-concrete floor and ceiling construction.

238,382.—W. Herbst: Construction of concrete floors.

38,407.—G. V. Evers: Cement kilns. 2

238,452.—L. Robertson: Reinforced-concrete linings for window and door openings.

238,605.—W. H. Coward: Concrete building.

238,625.—W. E. Clifton and Clifton-Ewart Construction Co., Ltd.: Concrete structures.

238,631.—M. Vogel-Jorgensen: Process of burning cement.

Trade Notices.

Concrete Floors.—We have received from the Trussed Concrete Steel Co., Ltd., of 22, Cranley Gardens, S.W.7, a copy of a well-produced brochure describing the "Truscon" floor. It is claimed that this floor is cheaper than timber for long spans and heavy loads, and that it is only 10 per cent. dearer than timber for short spans and light loads. The weight is only 34 lbs. per sq. ft. for 6-in. floors, 39 lbs. for 8-in. floors, and 44 lbs. for 10-in. floors. The brochure is fully illustrated with diagrams and photographs, the latter including some of the important buildings in which this floor has been used.

Concrete Mixers.—Two new catalogues (Nos. 39 and 40) have been issued by Messrs. Ransome Machinery Co. (1920), Ltd., of 46, Victoria Street, S.W.1. These both deal with concrete mixers, No. 39 illustrating the firm's Standard type machines and No. 40 the "Featherweight," "Lightweight," and Welterweight" machines, and they should be in the hands of all users of such plant. Each model is illustrated by photographs and diagrams, and useful working instructions are given. These machines have outputs of from 4 cu. ft. to 2 cu. yds. per batch of mixed materials, and may be had in a variety of patterns for different purposes.

Concrete-Breaking Cartridge.—Messrs. The Demolition and Construction Co., Ltd., of 73, Victoria St., S.W.1, have issued an interesting booklet describing the application of their pneumatic and hydraulic demolition plant, for use in cases where the use of explosives is impracticable. With the hydraulic cartridge, the disruptive effect is obtained by means of a number of small solid rams or presses which move out under hydraulic pressure at right-angles from the steel cylinder, and a pressure of 8 to 10 tons per sq. in. may be obtained. The method, which has the advantage that masses of concrete may be broken without vibration, has been used on a large number of contracts in this country.

Barrows.—A folder describing and illustrating "Titan" steel barrows has been issued by the manufacturers, Messrs. The Constructional Engineering Co., Ltd., of Charles Henry Street, Birmingham. The capacity of these barrows ranges from 2½ to 7½ cu. ft.

Wire Rope.—Messrs. Bruntons, of the Wire Mills, Musselburgh, have issued a booklet describing their new "Tru-lay" wire rope, and "True-loc" fittings. It is claimed that the new principle on which these ropes are made gives them a considerably longer life than the usual patterns, and that they are the only ropes which evenly balance the load on individual strands; they are claimed to be free from kinking, and easily spliced. "Tru-loc" fittings are, we are informed, more easily and quickly applied than zinc sockets, and give better equalisation of the loads on the wires.

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